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by

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and for minimizing the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This report documents the development and application of a real time automatic control model that can be used with a combined sewer system to minimize overflows during storm events. The results of this project will be useful to many communities facing problems of wet weather pollution of their receiving waters.

Francis T. Mayo, Director
Municipal Environmental
Research Laboratory

ABSTRACT

The primary objective of this study was to develop a real time automatic control model that could be used in connection with a combined sewer system to minimize overflows during storms. The model was applied to the North Shore Outfall Consolidation Project in San Francisco. This project consists of a large transport-storage facility that intercepts existing outfalls and allows flows to be pumped to a primary or a secondary treatment plant depending on operational strategy. Four reactive control strategies and one reactive-predictive strategy were formulated and evaluated for pollutant removal performance. The cost effectiveness of each was then determined.

This report was submitted in partial fulfillment of Cooperative Agreement No. CS 803743 by the San Francisco Wastewater Program of the City and County of San Francisco under the sponsorship of the U.S. Environmental Protection Agency. This report covers the period August 1975 to December 1981 and was completed September 1982.

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I. SUMMARY

This report presents the findings of the first phase of a research effort dealing with the development and application of concepts and methods for analyzing, evaluating and implementing real time automatic control (RTAC) for combined sewer systems. The North Shore Outfall Consolidation (NSOC) Project, presently under construction in the City of San Francisco, served as the demonstration vehicle for the study.

Following Chapter II, which provides the project background and presents the scope, objectives, and organization of the study, Chapter III describes the NSOC Project in terms of facilities and planned operation. The Project consists of a large transport-storage facility that intercepts combined wastewater flows at existing outfalls and delivers them to a pumping station that can direct the inflows to a secondary or primary treatment plant depending upon the operational strategy that is adopted. The size of the transport-storage facility is large enough to store a significant amount of wastewater for later treatment.

The subject of real time control is introduced in Chapter IV. The discussion covers the need for control, control objectives, methods of control, control philosophy, and the location and types of control devices, with the NSOC Project used for illustration.

The Control System objectives for the NSOC RTAC system were:

1. Limit untreated overflows to an average of less than one per year,
2. control the location of overflow points,
3. provide the best treatment possible to all flows,
4. make optimal use of all facilities to avoid higher costs, and
5. provide data logs and reports for current needs and for historical records.

To achieve these, three alternative methods of control were considered in the study; these were:

1. Remote manual control with data logging,
2. central digital (automatic) control and data logging, and
3. distributed digital control and data logging.

Only two types of control philosophies were applicable to the NSOC system; these were:

1. Purely reactive control, and
2. integrated reactive-predictive control.

Four reactive control strategies and one reactive-predictive strategy were formulated and evaluated in the course of the study; these were:

I Basic Operating Strategy*
II Basic Operating Strategy with Seasonal Constraints
III Basic Operating Strategy with Minimum Storage Constraints
IV Basic Operating Strategy with Seasonal and Minimum Storage Constraints
V Basic Operating Strategy with Rainfall Prediction and Seasonal and Minimum Storage Constraints

These strategies are first described generally and in terms of the basic philosophy of control they represent and the types of control devices required to implement them in the NSOC system. The devices include level and flow gates, dams, weirs, pumps and digital control and computer hardware.

Chapters V and VI present information regarding four mathematical models that are applicable to RTAC studies as analytical tools that could be used for developing and testing control strategies, and for simulating inflows to the system and its resulting hydraulic response, based either on historical or real time conditions. The three computer models covered in Chapter V are: TREAT, and the RUNOFF and TRANSPORT Blocks of EPA's Storm Water Management Model (SWMM). Each model is described briefly and its potential usefulness in RTAC studies is explored. Applications of the models to the NSOC Project are described and example results presented.

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*This strategy represents the basic required strategy to meet the overflow objective.

Chapter VI covers the subject of rainfall prediction, which is a necessary input for predictive control strategies. The subjects covered include the alternative techniques available for rainfall prediction, local rainfall characteristics and data availability in the San Francisco area, and the computer model RAFORT which was developed specifically for the NSOC Project and is applicable to real time operation. Examples of rainfall predictions made with RAFORT are presented.

Having considered the basic analytical tools necessary to assess the alternative control strategies for the NSOC Project, the results of the assessment are presented in Chapter VII. For each strategy introduced in Chapter IV, detailed simulations of the NSOC system over 70 years were carried out using the computer model TREAT. The major inputs and processes modeled included rainfall/runoff, dry weather flows, primary and secondary treatment, storage, and untreated overflows. The TREAT algorithms were modified to allow the operation of the control system for each alternative control strategy simulated. A complete statistical frequency distribution for twelve performance indicators were output by TREAT, including such measures as the annual number, amount and duration of treatment, storage, and overflows. A rough measure of the relative reductions in pollutant load discharged in each case was also computed.

The annual capital and operation and maintenance costs for each strategy were estimated based on the added NSOC costs attributed to the control system. A cost-effectiveness analysis was then performed to arrive at conclusions on the best strategies for the NSOC RTAC system.

The conclusions and recommendations presented relate to the general concepts and approaches developed during the study rather than to the specific operation of the San Francisco NSOC Project. Some of the major conclusions drawn were:

- Control strategies can have a significant impact on the overall performance of the system and should be considered during the facilities planning and design phase.
- For the NSOC Project and an overflow objective of one per year, the best control strategy appears to be reactive.
- Reactive-predictive strategies have modest potential for improving system performance, but are limited by inaccuracies in rainfall prediction methodology.
- The model TREAT is useful in evaluating control strategies but is inadequate for use in real time control if a predictive strategy is selected.
- The RUNOFF Block of SWMM could be used in real time control under a predictive strategy.
- The TRANSPORT Block of SWMM is too complex and is too subject to numerical instabilities for use in real time control.
- The best control strategy is dependent upon the system design and the control objectives; changes in either will alter the control strategy that provides the best performance.

II. INTRODUCTION

BACKGROUND

In 1971, the City of San Francisco adopted their Master Plan for Waste Water Management. That plan was subsequently updated by a supplement in 1973 and described in a final EIR dated 1974. It described several systems of facilities to reduce City-wide untreated overflows from an existing statistical average of 82 per year to four alternative levels, 0.2, 1, 4 or 8 per year. The latest facilities planning efforts estimate the cost for wet weather facilities will range from approximately $1.3 billion for the 1 overflow/year system to $0.8 billion for an 8 overflow per year system. The City's recommendation was for 8 overflows per year. Regulatory agency requirements stipulated in NPDES Permits varied from 1 to 4 overflows per year depending on the area of the city. A portion of the facilities needed to achieve these requirements are now in design or under construction. This report describes a research project, funded by the U.S. Environmental Protection Agency, to evaluate alternative concepts and strategies for real time automatic control of the City's wet weather facilities. The North Shore region of the City was used as a demonstration area.

SCOPE AND OBJECTIVES

The primary objective of this study is to develop and demonstrate the application of a real time automatic control (RTAC) model for combined sewer systems. The development work will include modification of existing simulation models for operation of real time control systems and interfacing between the simulation models and the control model. Demonstration of the RTAC system is to be for the North Shore Outfall Consolidation Project in the City and County of San Francisco. In addition, there are several secondary objectives that will make this project generally useful to other cities contemplating RTAC systems for control of combined sewer flows. These are:

1. To develop the control model software in Fortran computer language and in a sufficiently general manner if possible so that it can be applied to other cities without major modification.
2. To perform a cost-effectiveness analysis on alternative control strategies to determine the best RTAC system for controlling overflows from the North Shore System.
3. To describe the step-by-step procedure for adapting the general control model to a specific prototype and for installing these models on the control system hardware.
4. To demonstrate the operation of the North Shore RTAC System.

The technical work required to develop and implement the RTAC system for the North Shore Project was divided into the following six major categories:

1. Control System Design
2. Control Model Development
3. Simulation Model Development
4. Rainfall Model Development
5. Control System Implementation
6. Project Management and Reporting

The overall project was also separated into three major phases of work. Within each phase, work would be accomplished on each of the six technical work areas listed above. Brief descriptions of the three phases of work are contained in the following paragraphs.

Phase I: Project Planning and Models Modification

1. Data Collection and Evaluation. Sufficient historical data shall be evaluated for selected runoff events to compute hourly, daily, and seasonal patterns of dry weather flow and quality and to determine relationships between wet weather flow and quality.

2. Define Control Objectives. Both lower control levels (individual subbasin optimizations) and higher level (master problem) control strategies shall be studied and developed simultaneously, then expanding the first catchment to the entire sewerage system. The entire sewerage system shall cover the entire demonstration area only.

Phase II: Software Development

1. Modify Simulation Models to Operate in Real Time. The following EPA sponsored projects shall be considered for evaluation:

- "Combined Sewer Overflow Abatement Program," Rochester, New York (Project No. Y-005141).
- "A Guide for Comprehensive Planning for Control of Urban Storm and Sanitary Flows," University of Florida (Project No. R802411).

2. Develop Real Time Control Model. The main function of the control model shall be to successively update control decisions as a storm progresses. This shall have automatic operation of control units and a closed-loop model control system comprised of real time feedback information to the rainfall-runoff-transport simulation model. Equations for control elements shall include regulator gates, pumping stations, perpendicular and sidespill overflow weirs, orifice underflows, tide gates, inverted siphons, etc. The hydraulic equations for these facilities shall consider both upstream and downstream conditions.

3. Develop Rainfall Prediction Model. A stochastic model shall be developed to predict rainfall patterns and intensity.

Phase III: Application and Demonstration

1. The Grantee shall be responsible for supply, construction, and operation of the telemetry (including sensors) and computer hardware system.

2. Calibration and Testing of Developed/Modified Models with Historical Data. The interfaced real time control and simulation models shall be tested with historical data to calibrate model parameters such as catchment storm runoff and quality coefficients, regulator discharge coefficients, etc.

3. Cost Effective Analysis. A cost effective analysis shall be performed on automatic real-time control versus alternative strategies. The following items shall be described in detail:

- The basis of measurement for the cost effective analysis.
- How a single basin is used to represent a total basin, and the assumptions made for the single basin with respect to the balance of the system.

This task will be contingent upon availability of funds during the period of post construction evaluation under this grant.

4. Demonstration of the Real Time Automatic Control. It is essential that a sufficient amount of data be taken during Task III-7 of the Grant Application to show conclusively the effectiveness of the control system developed in the earlier phases of the project. To assure that these data can be taken within the budget proposed, the Grantee shall use special care in managing the manpower and other resources for the task. To the maximum degree possible, these resources shall be utilized only during and following periods of precipitation. Expenditures during dry weather flow shall be very strictly limited.

The scope of work was subsequently refined through the development of a detailed plan of study which was the first task in the project. In addition, changes in the scope of work were required as the study progressed as a result of changes to the design of the North Shore Outfall Consolidation Project and in response to findings made during the conduct of Phase I.

PROJECT ORGANIZATION

The project was conducted for the Storm and Combined Sewer Section of the Municipal Environmental Research Laboratory, EPA, by the San Francisco Wastewater Program, City and County of San Francisco and their consultants Christos A. Phanartzis and Water Resources Engineers/Camp Dresser & McKee Inc. The individuals involved in this study were:

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City and County of San Francisco

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Gene T. Handa
David A. Jones
Thomas F. Landers
Christos A. Phanartzis, Consulting Hydrologist

Water Resources Engineers/Camp Dresser & McKee Inc.

Donald E. Evenson, Co-Project Manager
Larry A. Roesner, Co-Project Manager
Paul R. Giguere
John D. Manning

In addition, many other individuals contributed helpful suggestions and constructive criticisms throughout the study.

III. NORTH SHORE OUTFALL CONSOLIDATION PROJECT

GENERAL DESCRIPTION

The prototype facility which will serve as the demonstration vehicle for the RTAC project is situated in the North Shore area of the City of San Francisco. The proposed North Shore Outfall Consolidation (NSOC) Project, when completed, will be an integral part of the collection, transport, outfall, and treatment plant system which serves the North Point drainage district. The NSOC Project is also one of the major elements in the City of San Francisco's Master Plan for Wastewater Management, the major components of which are shown in Figure III-1.

The NSOC Project consists of a large transport-storage system located along the periphery of the northeastern shoreline of the City to intercept both dry and wet weather flows. The Master Plan calls for the transport of wet weather flows to an existing primary treatment plan (North Point Water Pollution Control Plant; hereinafter referred to as the North Point Plant) while the dry weather flows will be pumped to a secondary treatment plant in the Southeastern region of the City. The project also includes a major new pumping facility (the North Shore Pump Station) to lift the wastewater from the storage-transport facility to the treatment plants. The storage-transport facility will be large enough to store storm flows until treatment capacity is available and thus will reduce the volume of untreated combined sewage discharges to San Francisco Bay during storm periods. The facilities have been sized to reduce the average number of untreated overflows per year from approximately 44 to less than 1.* However, the number of overflows, the location of overflows and the degree of treatment given to the wastewater, is dependent on the control devices and strategies used to operate the system. The NSOC Project is expected to cost $68.7 million and will require approximately three years to complete. As with other elements of the Master Plan, the NSOC

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*A recent decision by the California Regional Water Quality Control Board - San Francisco Region relaxed the discharge requirement to four (4) per year. Some design changes may occur as a result of this decision.

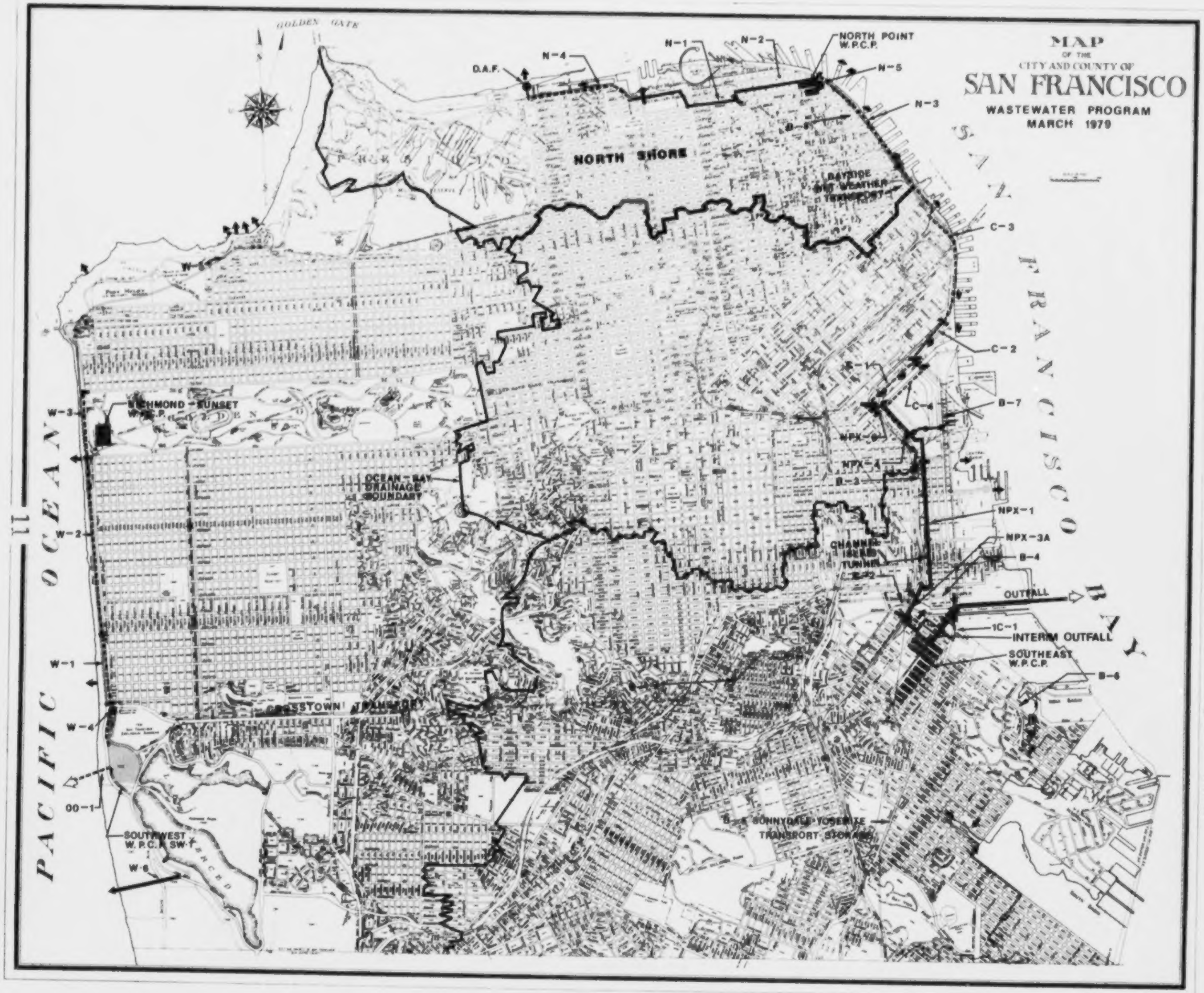

MAP OF THE CITY AND COUNTY OF SAN FRANCISCO
WASTEWATER PROGRAM
MARCH 1979
GOLDEN GATE
SAN FRANCISCO BAY
PACIFIC OCEAN
NORTH SHORE
NORTH POINT W.P.C.P.
D.A.F.
N-1
N-2
N-3
N-4
N-5
BAYSIDE WET WEATHER TRANSPORT
C-2
C-3
C-4
B-7
B-3
B-4
B-6
NPX-1
NPX-3A
NPX-6
OUTFALL
IC-1
INTERIM OUTFALL
SOUTHEAST W.P.C.P.
SUNNYDALE-YOSEMITE TRANSPORT STORAGE
OCEAN-BAY DRAINAGE BOUNDARY
CHANNEL-ISLAIS TUNNEL
RICHMOND-SUNSET W.P.C.P.
CROSSTOWN TRANSPORT
SOUTHWEST W.P.C.P. SW-1
OO-1
W-1
W-2
W-3
W-4
W-5
W-6

FIGURE III-1

MAJOR ELEMENTS IN SAN FRANCISCO'S MASTER PLAN FOR WASTEWATER MANAGEMENT

MAP REF	PROPOSED CONSTRUCTION CONTRACTS AND SUPPORTING DOCUMENTS
	NORTH POINT CROSSTOWN
NPX-1	INDIANA STREET FORCE MAIN
NPX-3A	FORCE MAIN EVANS TO ARMY
NPX-3B	FORCE MAIN
NPX-4	FORCE MAIN, PUMP STATION TO INDIANA
NPX-6	CHANNEL PUMP STATION
NPX-6A	FORCF MAIN, PUMP STATION TO HOWARD
	NORTH SHORE OUTFALLS CONSOLIDATION
N-1	FORT MASON TUNNEL
N-2	NORTH POINT STREET
N-3	NORTH EMBARCADERO
N-4	MARINA
N-5	NORTH SHORE PUMP STATION
	BAYSIDE FACILITIES
B-1	CROSSTOWN TUNNEL
B-2	ISLAIS CREEK PUMP STATION
B-3	CHANNEL-ISLAIS LOW LEVEL TUNNEL
B-4	ISLAIS CREEK TRANSPORT STORAGE
B-5	SUNNYDALE YOSEMITE TRANSPORT STORAGE
B-6	HUNTERS POINT FACILITIES
B-7	MARIPOSA FACILITIES
B-8	NORTH SHORE WET WEATHER FORCE MAIN
	CHANNEL OUTFALL CONSOLIDATION
C-1	BERRY
C-2	KING
C-3	SOUTH EMBARCADERO
C-4	SOUTH SIDE
	WEST SIDE TRANSPORT
W-1	PUMP STA. TO NORIEGA
W-2	NORIEGA TO LINCOLN
W-3	LINCOLN TO FULTON
W-4	WST PUMP STA. & F.M.
W-5	RICHMOND TRANSPORT
W-6	LAKE MERCED TRANSPORT (TUNNEL)
	ISLAIS CREEK OUTFALLS CONSOLIDATION
IC-1	ISLAIS CREEK SOUTH SIDE
OO-1	SOUTHWEST OCEAN OUTFALL
SE-4	SOUTHEAST WATER POLLUTION CONTROL PLANT
SW-1	SOUTHWEST WATER POLLUTION CONTROL PLANT

LEGEND

┄┄┄	UNDER DESIGN
━━━	DESIGNED
┄┄┄	UNDER CONSTRUCTION
━━━	CONSTRUCTED
───	TENTATIVE CONCEPTS*
━•━	EXISTING BOUNDARY BETWEEN NORTH POINT AND SOUTHEAST DISTRICTS
→	OUTFALLS
●	PUMPING STATION

Project has been planned to be functional upon completion, and to provide for immediate improvements to existing receiving water conditions. The operation of the system will then be altered as other Master Plan elements are constructed and placed in operation.

STORAGE-TRANSPORT SYSTEM

The alignment of the NSOC transport-storage facility is shown in Figure III-2. The facility commences at the Baker Street Dissolved Air Flotation Treatment Facility (DAF) and extends east along Marina Boulevard to Laguna Street. This segment is seaward of the last sewer connection and as far north as an existing seawall permits. The alignment then heads east through Fort Mason, exiting Fort Mason near the intersection of Bay Street and Van Ness Avenue, and continues along Bay Street to Leavenworth Street, then north on Leavenworth Street to North Point Street and along North Point to the proposed North Shore Pump Station at Kearny Street. The alignment is joined by a 7-foot sewer in Powell Street between Beach and North Point Streets. The other major segment extends along The Embarcadero from Jackson Street to the North Shore Pump Station.

Figure III-3 is a perspective as viewed from the Bay which shows the interrelationship of the NSOC Project with the existing sewerage system. Also shown in Figure III-3 are typical cross-sections of the transport-storage facility. The preliminary profile of the NSOC Project is depicted in Figure III-4. The dry weather flow channel and the slope of the transport-storage facility are designed to provide a velocity of at least 2.5 fps during minimum flow. The total storage volume in the facility is 27.4 million gallons* which is equivalent to the runoff produced by 0.64 inches of rain over the tributary area of 2,247 acres (translated to 1,576 equivalent acres by runoff coefficients).

---

*Subsequently redesigned with 19.0 MG of storage to reflect the change in allowable overflows from one per year to four per year.

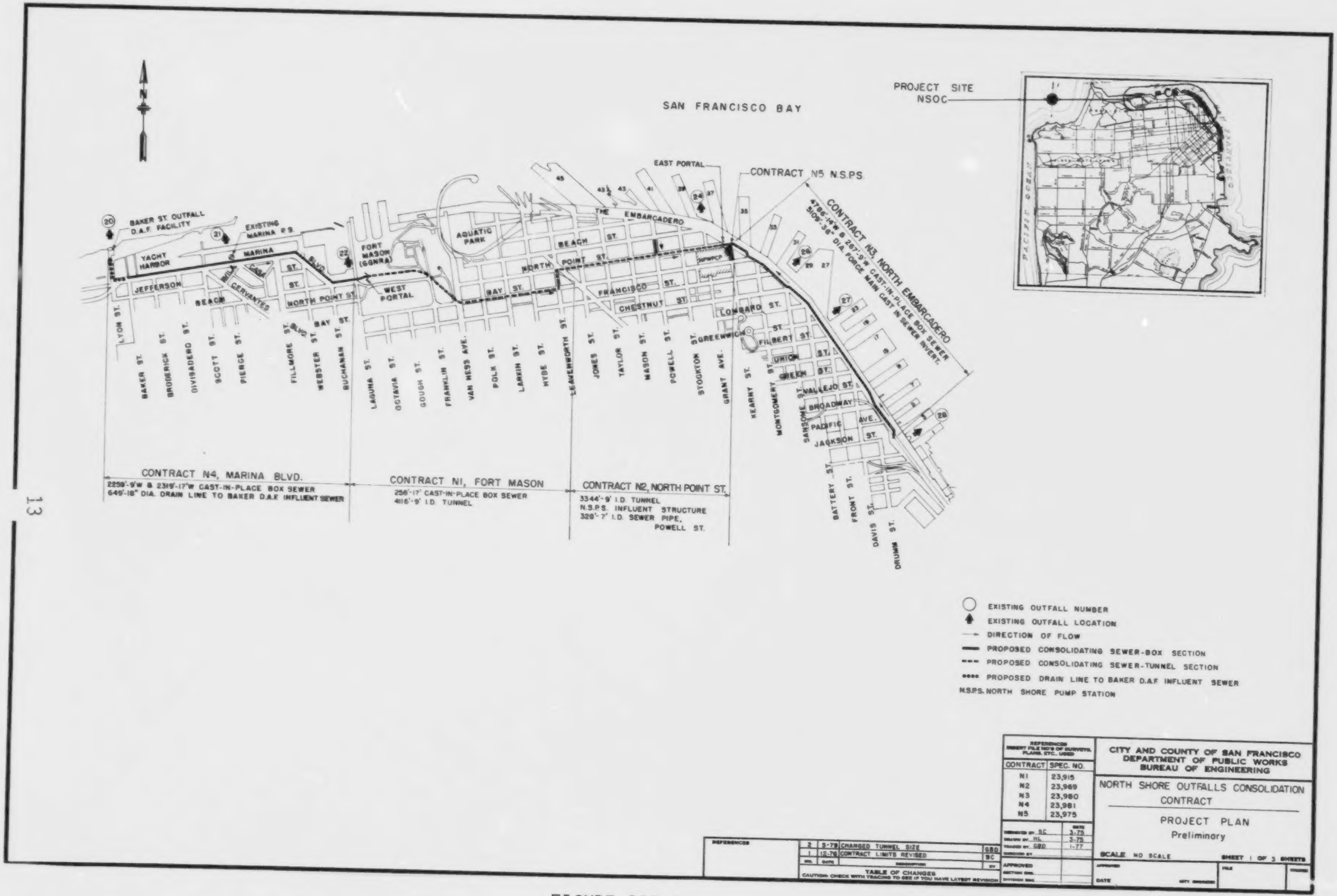
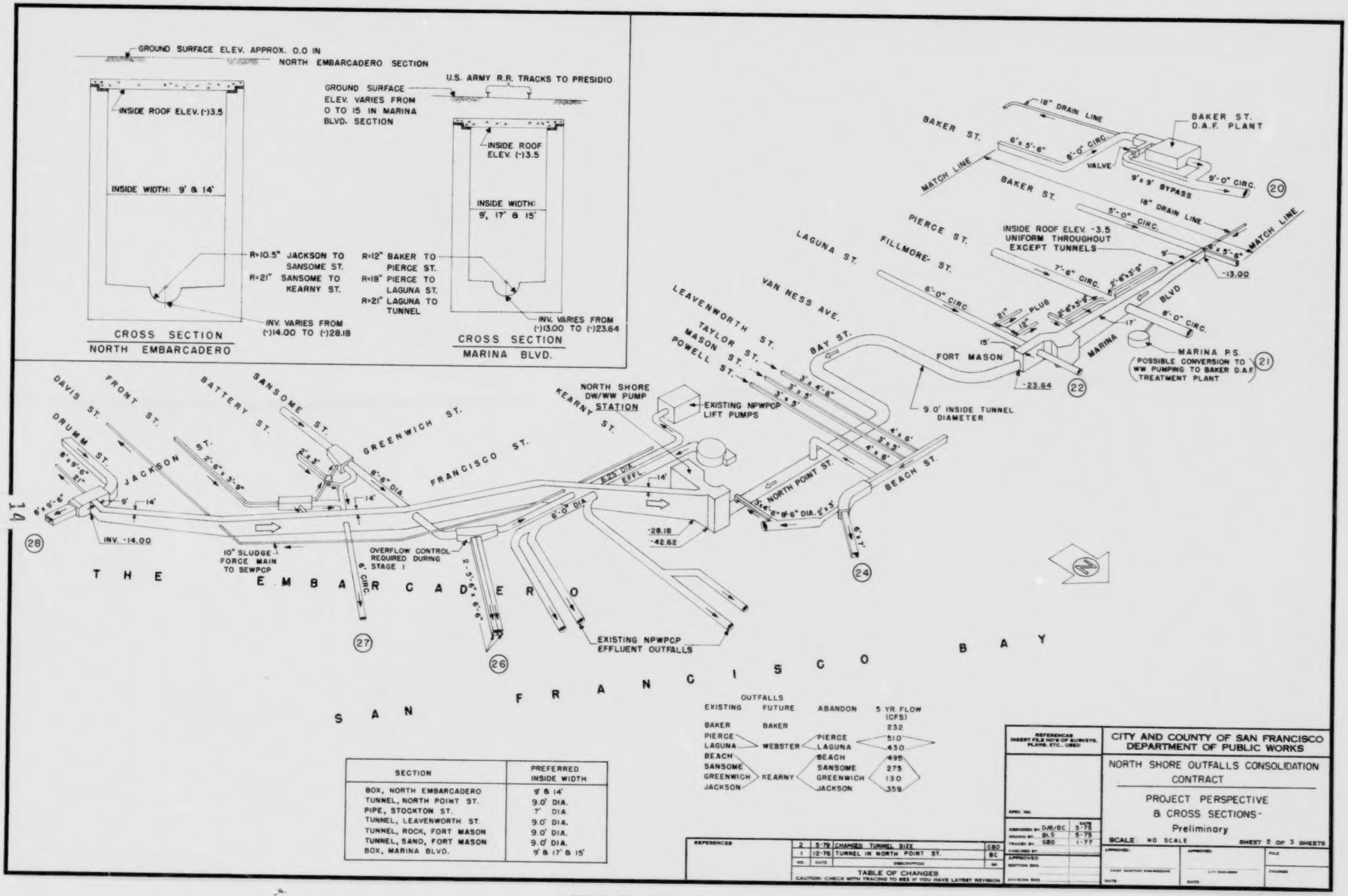

N
SAN FRANCISCO BAY
PROJECT SITE
NSOC
PACIFIC OCEAN
SAN FRANCISCO
20
BAKER ST. OUTFALL
D.A.F. FACILITY
21
EXISTING
MARINA P.S.
22
FORT
MASON
(GGNRA)
WEST
PORTAL
AQUATIC
PARK
YACHT
HARBOR
MARINA
BLVD.
JEFFERSON
BEACH
CERVANTES
CASA
ST.
NORTH POINT ST.
BAY ST.
THE EMBARCADERO
BEACH ST.
NORTH POINT ST.
FRANCISCO ST.
CHESTNUT ST.
LOMBARD ST.
GREENWICH
FILBERT ST.
UNION ST.
GREEN ST.
VALLEJO ST.
BROADWAY
PACIFIC AVE.
JACKSON ST.
NPWPCP
EAST PORTAL
CONTRACT N5 N.S.P.S.
CONTRACT N3, NORTH EMBARCADERO
4786'-14'W & 267'-9'W CAST-IN-PLACE BOX SEWER
5109'-36" DIA. FORCE MAIN CAST IN SEWER INVERT.
45
43½
43
41
39
24
37
35
33
31
26
29
27
27
23
19
17
9
7
5
3
1
28
LYON ST.
BAKER ST.
BRODERICK ST.
DIVISADERO ST.
SCOTT ST.
PIERCE ST.
FILLMORE ST.
WEBSTER ST.
BUCHANAN ST.
LAGUNA ST.
OCTAVIA ST.
GOUGH ST.
FRANKLIN ST.
VAN NESS AVE.
POLK ST.
LARKIN ST.
HYDE ST.
LEAVENWORTH ST.
JONES ST.
TAYLOR ST.
MASON ST.
POWELL ST.
STOCKTON ST.
GRANT AVE.
KEARNY ST.
MONTGOMERY ST.
SANSOME ST.
BATTERY ST.
FRONT ST.
DAVIS ST.
DRUMM ST.
CONTRACT N4, MARINA BLVD.
2259'-9'W & 2319'-17'W CAST-IN-PLACE BOX SEWER
649'-18" DIA. DRAIN LINE TO BAKER D.A.F. INFLUENT SEWER
CONTRACT N1, FORT MASON
258'-17' CAST-IN-PLACE BOX SEWER
4116'-9' I.D. TUNNEL
CONTRACT N2, NORTH POINT ST.
3344'-9' I.D. TUNNEL
N.S.P.S. INFLUENT STRUCTURE
328'-7' I.D. SEWER PIPE,
POWELL ST.
○ EXISTING OUTFALL NUMBER
EXISTING OUTFALL LOCATION
DIRECTION OF FLOW
PROPOSED CONSOLIDATING SEWER-BOX SECTION
PROPOSED CONSOLIDATING SEWER-TUNNEL SECTION
PROPOSED DRAIN LINE TO BAKER D.A.F. INFLUENT SEWER
N.S.P.S. NORTH SHORE PUMP STATION
REFERENCES
INSERT FILE NO'S OF SURVEYS, PLANS, ETC., USED
CONTRACT | SPEC. NO.
N1 | 23,915
N2 | 23,969
N3 | 23,980
N4 | 23,981
N5 | 23,975
DESIGNED BY SC 3-75
DRAWN BY HL 3-75
TRACED BY GBD 1-77
CHECKED BY
APPROVED
SECTION ENG.
DIVISION ENG.
CITY AND COUNTY OF SAN FRANCISCO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING
NORTH SHORE OUTFALLS CONSOLIDATION
CONTRACT
PROJECT PLAN
Preliminary
SCALE NO SCALE
SHEET 1 OF 3 SHEETS
APPROVED
DATE
CITY ENGINEER
FILE
CHANGE
REFERENCES
2 | 3-79 | CHANGED TUNNEL SIZE | GBD
1 | 12-76 | CONTRACT LIMITS REVISED | BC
NO. | DATE | DESCRIPTION | BY
TABLE OF CHANGES
CAUTION: CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION

FIGURE III-2

PRELIMINARY ALIGNMENT OF THE TRANSPORT-STORAGE FACILITY OF THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT


GROUND SURFACE ELEV. APPROX. 0.0 IN
NORTH EMBARCADERO SECTION
INSIDE ROOF ELEV. (-)13.5
INSIDE WIDTH: 9' & 14'
R=10.5" JACKSON TO SANSOME ST.
R=21" SANSOME TO KEARNY ST.
INV. VARIES FROM (-)14.00 TO (-)28.18
CROSS SECTION
NORTH EMBARCADERO
U.S. ARMY R.R. TRACKS TO PRESIDIO
GROUND SURFACE ELEV. VARIES FROM 0 TO 15 IN MARINA BLVD. SECTION
INSIDE ROOF ELEV. (-)13.5
INSIDE WIDTH: 9', 17' & 15'
R=12" BAKER TO PIERCE ST.
R=18" PIERCE TO LAGUNA ST.
R=21" LAGUNA TO TUNNEL
INV. VARIES FROM (-)13.00 TO (-)23.64
CROSS SECTION
MARINA BLVD.
18" DRAIN LINE
BAKER ST.
6' x 5'-6"
6'-0" CIRC.
VALVE
BAKER ST. D.A.F. PLANT
9' x 9' BYPASS
9'-0" CIRC.
20
MATCH LINE
BAKER ST.
18" DRAIN LINE
5'-0" CIRC.
PIERCE ST.
INSIDE ROOF ELEV. -3.5 UNIFORM THROUGHOUT EXCEPT TUNNELS
9'
6' x 5'-6"
MATCH LINE
-13.00
LAGUNA ST.
FILLMORE- ST.
7'-6" CIRC.
2'-6" x 3'-9"
VAN NESS AVE.
6'-0" CIRC.
21"
PLUG
12"
2'-6" x 3'-9"
BLVD
8'-0" CIRC.
17'
MARINA
MARINA P.S.
(POSSIBLE CONVERSION TO WW PUMPING TO BAKER D.A.F. TREATMENT PLANT)
21
LEAVENWORTH ST.
TAYLOR ST.
MASON ST.
POWELL ST.
BAY ST.
15'
FORT MASON
-23.64
22
3' x 4'-6"
3' x 5'
3' x 5'
9.0' INSIDE TUNNEL DIAMETER
NORTH SHORE DW/WW PUMP STATION
EXISTING NPWPCP LIFT PUMPS
4' x 6'
3' x 3'
4' x 6'
BEACH ST.
NORTH POINT ST.
3' x 4'-6" 8'-6" DIA. 2' x 3'
6' x 7'
24
DAVIS ST.
FRONT ST.
BATTERY ST.
SANSOME ST.
DRUMM ST.
6' x 9'-6"
21"
JACKSON ST.
2'-6" x 3'-9"
2' x 3'
GREENWICH ST.
8'-6" DIA.
FRANCISCO ST.
KEARNY ST.
6.25' DIA. EFFL.
14'
6' x 9'-6"
9'
14'
14'
INV. -14.00
28
10" SLUDGE FORCE MAIN TO SEWPCP
6' CIRC.
27
OVERFLOW CONTROL REQUIRED DURING STAGE 1
2-5'-6" x 6'-6"
26
8'-0" DIA.
-28.18
-42.62
EXISTING NPWPCP EFFLUENT OUTFALLS
THE EMBARCADERO
SAN FRANCISCO BAY
N
OUTFALLS
EXISTING
FUTURE
ABANDON
5 YR FLOW (CFS)
BAKER
BAKER
232
PIERCE
LAGUNA
WEBSTER
PIERCE
LAGUNA
510
430
BEACH
SANSOME
GREENWICH
JACKSON
KEARNY
BEACH
SANSOME
GREENWICH
JACKSON
495
275
130
359
REFERENCES
2 5-79 CHANGED TUNNEL SIZE GBD
1 12-76 TUNNEL IN NORTH POINT ST. BC
NO. DATE DESCRIPTION BY
TABLE OF CHANGES
CAUTION: CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION
REFERENCES INSERT FILE NO'S OF SURVEYS, PLANS, ETC., USED
DESIGNED BY DJB/BC 5-75
DRAWN BY BLS 5-75
TRACED BY GBD 1-77
CHECKED BY
APPROVED
SECTION ENG.
DIVISION ENG.
CITY AND COUNTY OF SAN FRANCISCO
DEPARTMENT OF PUBLIC WORKS
NORTH SHORE OUTFALLS CONSOLIDATION CONTRACT
PROJECT PERSPECTIVE & CROSS SECTIONS - Preliminary
SCALE: NO SCALE
SHEET 2 OF 3 SHEETS
APPROVED
CHIEF SANITARY ENGINEERING
DATE
APPROVED
CITY ENGINEER
DATE
FILE
CHANGE

SECTION	PREFERRED INSIDE WIDTH
BOX, NORTH EMBARCADERO	9' & 14'
TUNNEL, NORTH POINT ST.	9.0' DIA.
PIPE, STOCKTON ST.	7' DIA.
TUNNEL, LEAVENWORTH ST.	9.0' DIA.
TUNNEL, ROCK, FORT MASON	9.0' DIA.
TUNNEL, SAND, FORT MASON	9.0' DIA.
BOX, MARINA BLVD.	9' & 17' & 15'

FIGURE III-3

PRELIMINARY PROJECT PERSPECTIVE AND CROSS-SECTIONS OF THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT

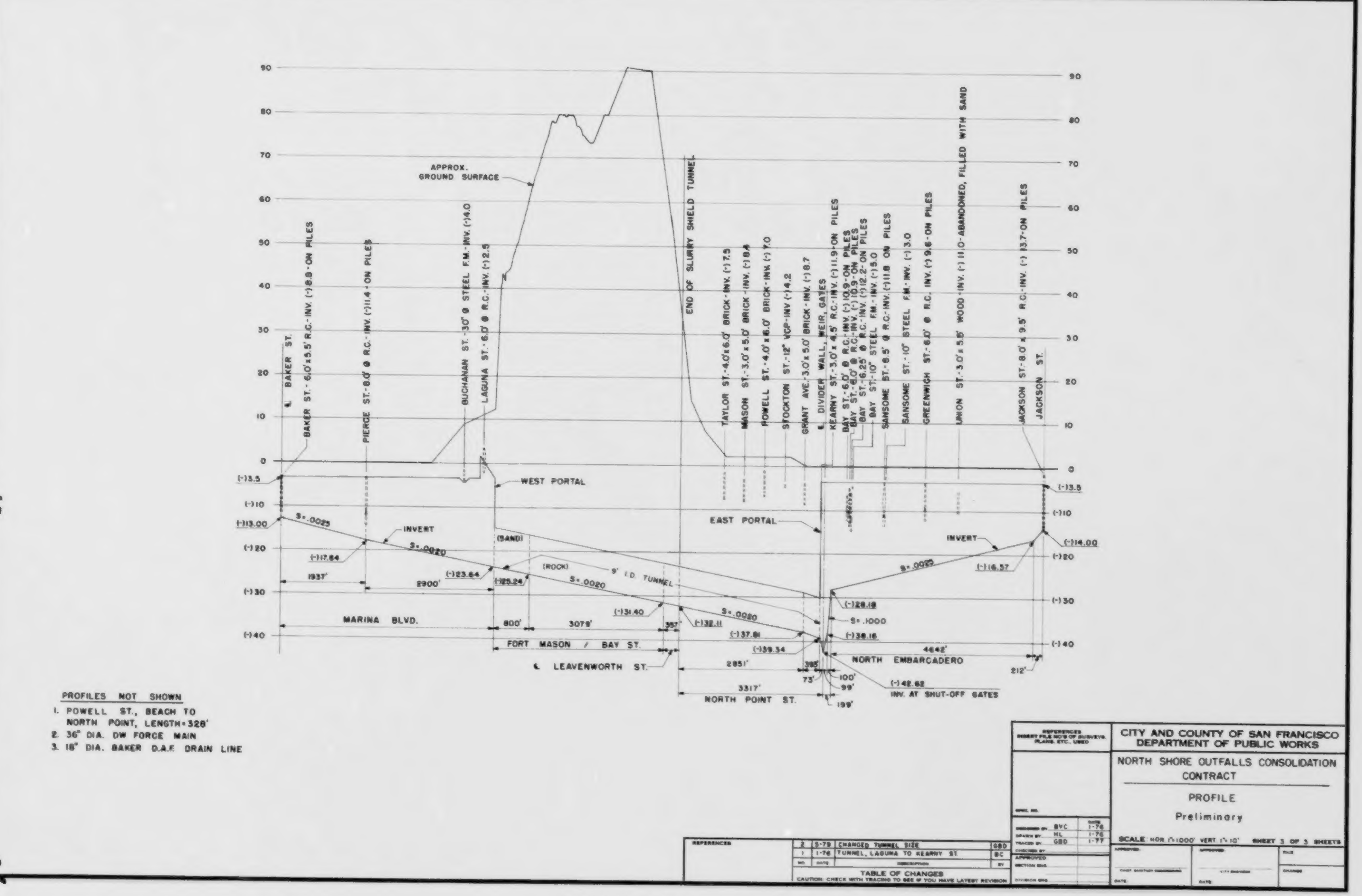

90
80
70
60
50
40
30
20
10
0
(-)13.5
(-)10
(-)13.00
(-)20
(-)30
(-)40
APPROX. GROUND SURFACE
℄ BAKER ST.
BAKER ST.-6.0'x5.5' R.C.-INV. (-)8.8-ON PILES
PIERCE ST.-8.0' ⌀ R.C.-INV. (-)11.4-ON PILES
BUCHANAN ST.-30" ⌀ STEEL F.M.-INV. (-)4.0
LAGUNA ST.-6.0' ⌀ R.C.-INV. (-)2.5
WEST PORTAL
END OF SLURRY SHIELD TUNNEL
TAYLOR ST.-4.0'x6.0' BRICK-INV. (-)7.5
MASON ST.-3.0'x5.0' BRICK-INV. (-)8.4
POWELL ST.-4.0'x6.0' BRICK-INV. (-)7.0
STOCKTON ST.-12" VCP-INV (-)4.2
GRANT AVE.-3.0'x5.0' BRICK-INV. (-)8.7
℄ DIVIDER WALL, WEIR, GATES
KEARNY ST.-3.0'x4.5' R.C.-INV. (-)11.9-ON PILES
BAY ST.-6.0' ⌀ R.C.-INV. (-)10.9-ON PILES
BAY ST.-6.0' ⌀ R.C.-INV. (-)10.9-ON PILES
BAY ST.-6.25' ⌀ R.C.-INV. (-)12.2-ON PILES
BAY ST.-10" STEEL F.M.-INV. (-)5.0
SANSOME ST.-6.5' ⌀ R.C.-INV. (-)11.8 ON PILES
SANSOME ST.-10" STEEL F.M.-INV. (-)3.0
GREENWICH ST.-6.0' ⌀ R.C. INV. (-)9.6-ON PILES
UNION ST.-3.0'x5.5' WOOD-INV. (-)11.0-ABANDONED, FILLED WITH SAND
JACKSON ST.-8.0'x9.5' R.C.-INV. (-)13.7-ON PILES
JACKSON ST.
EAST PORTAL
S=.0025
INVERT
S=.0020
(-)17.84
(SAND)
(ROCK)
9' I.D. TUNNEL
(-)23.64
(-)25.24
S=.0020
1937'
2900'
(-)31.40
(-)32.11
S=.0020
MARINA BLVD.
800'
3079'
357'
FORT MASON / BAY ST.
℄ LEAVENWORTH ST.
(-)37.81
(-)39.34
2851'
385'
73'
3317'
NORTH POINT ST.
(-)28.19
S=.1000
(-)38.16
100'
99'
199'
(-)42.62
INV. AT SHUT-OFF GATES
NORTH EMBARCADERO
4642'
212'
INVERT
S=.0025
(-)16.57
(-)14.00
PROFILES NOT SHOWN
1. POWELL ST., BEACH TO NORTH POINT, LENGTH=328'
2. 36" DIA. DW FORCE MAIN
3. 18" DIA. BAKER D.A.F. DRAIN LINE
REFERENCES
2 | 5-79 | CHANGED TUNNEL SIZE | GBD
1 | 1-76 | TUNNEL, LAGUNA TO KEARNY ST. | BC
NO. | DATE | DESCRIPTION | BY
TABLE OF CHANGES
CAUTION: CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION
REFERENCES INSERT FILE NO'S OF SURVEYS, PLANS, ETC., USED
DESIGNED BY BVC 1-76
DRAWN BY HL 1-76
TRACED BY GBD 1-77
CHECKED BY
APPROVED
SECTION ENG.
DIVISION ENG.
CITY AND COUNTY OF SAN FRANCISCO
DEPARTMENT OF PUBLIC WORKS
NORTH SHORE OUTFALLS CONSOLIDATION CONTRACT
PROFILE
Preliminary
SCALE: HOR. 1"=1000' VERT. 1"=10'
SHEET 3 OF 3 SHEETS
APPROVED:
CHIEF SANITARY ENGINEERING
CITY ENGINEER
DATE
FILE
CHANGE

FIGURE III-4

PRELIMINARY PROFILE OF THE
NORTH SHORE OUTFALL CONSOLIDATION PROJECT

NORTH SHORE PUMP STATION

The proposed North Shore Pump Station (NSPS) is located near the site of the existing North Point Plant (See Figures III-1 and III-2) and initially will serve to pump the combined wastewater out of the transport-storage facility to the influent line of the North Point Plant.* In later stages of the Master Plan development, dry weather flows from the North Shore area will be transported to an expanded Southeast Water Pollution Control Plant (hereinafter called the Southeast Plant) for secondary treatment while the wet weather flows will be pumped to the proposed Southwest Water Pollution Control Plant (hereinafter called the Southwest Plant) for primary treatment and ocean disposal. The NSPS will include both dry and wet weather pumps which will come on and off line as water levels rise and fall in the wet well. The design maximum dry weather capacity is 30 mgd and the design wet weather pumping capacity is 150 mgd, which corresponds to the maximum hydraulic capacity of the existing North Point Plant.

NORTH POINT WATER POLLUTION CONTROL PLANT

The North Point Plant is an existing primary treatment facility which has an average dry-weather flow capacity of 65 mgd and a hydraulic capacity of 150 mgd (equivalent to 0.15 in/hr of rain over the tributary area). Treatment consists of prechlorination, screening, grit removal, preaeration, primary sedimentation with chemical coagulation capability, and postchlorination. Lift pumps downstream of the grit chambers determine the hydraulic capability of the plant and operate independently from the proposed NSPS.

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*Current 201 facilities planning studies underway by the City may result in modifications to the staging and implementation of the Master Plan; the changes will not have any major effects upon the general procedures developed in this research effort.

When the expansion of the Southeast Plant and other transport facilities are completed, the Master Plan calls for the North Point Plant to function solely in wet weather; the dry weather flows are to be sent to the expanded Southeast Plant for secondary treatment. When the Southwest Plant and necessary transport facilities are completed in later stages of the Master Plan, the North Point Plant will be phased out altogether with wet weather flows to be sent to the Southwest Plant for treatment.

BAKER STREET DISSOLVED AIR FLOTATION FACILITY

This existing dissolved air flotation (DAF) treatment plant has a capacity of 12 mgd. It has been proposed to use this facility during wet weather only, either to treat flows that would otherwise overflow at this location or to supplement the other wet weather treatment plant.

Because the plant's intake is at a relatively high elevation in the system, it is likely that pumping will be required to get the flow through the plant. One possibility that has been evaluated is the use of the existing Marina Pump Station, together with piping modifications and a control gate in the NSOC transport-storage facility just downstream of the Pierce Street inflow to pump wet weather flows to the DAF plant. However, the required modifications are expensive and further studies will be needed to determine if the DAF facility can be a cost-effective wet weather facility.

OUTFALLS AND OVERFLOW LOCATIONS

Presently, the hydraulic capacities of the North Point Plant and other existing treatment plants in the City are exceeded during most wet weather periods and discharges of untreated, combined sewage occur from one or more of the overflow structures located around the periphery of the City. These existing overflow locations are shown in Figure III-2. Overflow points numbered 20 through 28 will be intercepted by the NSOC facilities.

It is estimated that overflows now occur at one or more of these overflow points (20-28) an average of about 44 times per year.

The existing overflow locations have been ranked in order of acceptability for continued use for the infrequent overflows from the NSOC sewer. Table III-1 shows the rankings. Operational plans call for overflows to occur sequentially from the best to the least acceptable locations.

TABLE III-1

OVERFLOW LOCATION RANKINGS FOR THE
NORTH SHORE OUTFALL CONSOLIDATION PROJECT

Ranking*	Overflow Number**	Overflow Name
1	20	Baker
2	24	Beach
3	26	Sansome
4	28	Jackson
5	21	Pierce
6	22	Laguna
7	27	Greenwich

*A ranking of 1 means most desirable for use as an overflow point
**See Figures III-2 and III-3

The best overflow point is the dissolved air flotation facility bypass near Baker Street (No. 20). The discharge is the deepest of the existing outfalls, the closest to the Golden Gate entrance to the San Francisco Bay, and is in a high current zone. The Beach Street (No. 24), Sansome Street (No. 26), and Jackson Street (No. 28) outfalls are the next most desirable locations; the order determined by their proximity to the Golden Gate. The Pierce Street (No. 21) and Laguna Street (No. 22) outfalls discharge into or near boat berthing facilities and are not desireable locations for overflows.

SYSTEM OPERATION

The preceeding descriptions have dealt primarily with the physical facilities that constitute the NSOC Project. It is now appropriate to briefly discuss the planned operations of these facilities as called for in various stages of the City-wide Master Plan. These planned operations are referred to as the "Basic Operating Strategy" and are described in more detail in the "Transport-Storage Operational Plan Report" prepared by the City in June 1976. However, since 1976 the Master Plan has undergone some major revisions which affect the manner in which the system will operate. As a result, the City is now revising their 1976 Operational Plan Report. The operational plan for the NSOC Project that was considered most likely in 1977-78 was used in this research project.

According to the thinking at that time, the NSOC Project would operate during three distinct stages of master plan development in both dry and wet weather modes.

Stage I - Before the Southeast Plant is expanded
Stage II - After the Southeast Plant is expanded
Stage III - After the crosstown tunnel and Southwest Plant is constructed

The proposed operations of the system for three stages of development are described below. Table III-2 presents a summary of the system operation in terms of treatment and overflow priorities for all three stages.

TABLE III-2

OVERFLOW PRIORITY TABLE FOR THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT

		EXISTING CONDITION (STAGE I)	DEVELOPMENT PHASE	
			INTERIM (STAGE II)	ULTIMATE (STAGE III)
Flow Assignment		DW to NP WW to NP	DW to SE WW to NP	DW to SE WW to SW
Priority	Action			
1	Treat	DW to NP	30 MGD to SE[a]	30 MGD to SE[a]
2	"	WW to NP	150 MGD to NP	150 MGD to SW
3	"	WW to DAF	12 MGD to DAF	12 MGD to DAF
4	Overflow	Existing	DAF + Bypass	DAF + Bypass
5	"	Outlets	Beach Street[b]	Deep Water - Webster Street[c]
6	"	"	Sansome Street[b]	Deep Water - Kearny Street[c]
7	"	"	Jackson Street[b]	Beach Street[b]
8	"	"	Laguna Street[b]	Sansome Street[b]
9	"	"	Pierce Street[b]	Jackson Street[b]

(a) Will not continue pumping if rain occurs in Channel Street or Southeast Tributary Areas.

(b) Subject to elimination after several test years.

(c) If studies indicate cost effectiveness.

Dry Weather Operation

The average ultimate dry weather flow from the North Shore area is estimated at 21 mgd with a peak dry weather flow of 28.8 mgd. Table III-3 shows both the minimum and peak dry weather flow estimates by tributary area.

TABLE III-3

ULTIMATE DRY WEATHER FLOW ESTIMATES FOR THE
NORTH SHORE AREA, CITY OF SAN FRANCISCO

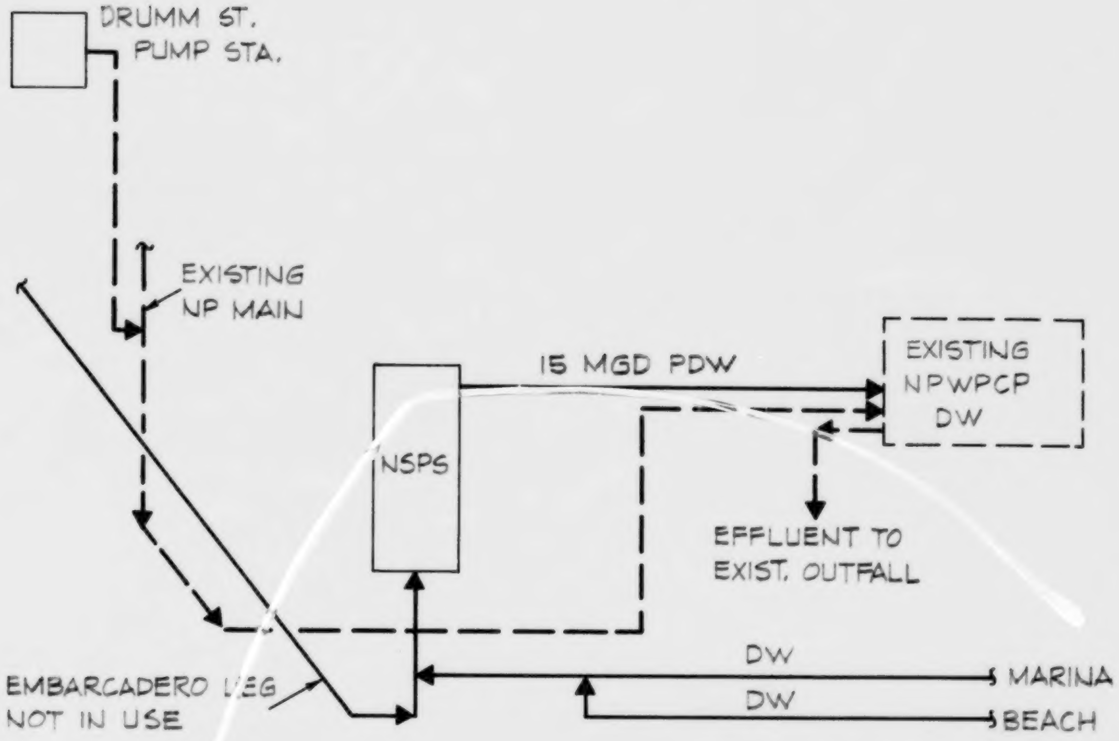
Tributary Area	Dry Weather Flows (mgd)		
	Minimum	Average	Peak
Marina	4.0	6.3	10
Beach	1.2	3.7	5.4
Embarcadero	2.9	7.7	13.4
TOTALS	8.1	17.7	28.8
Design Criteria	7		30

Stage I

This stage represents the period when the NSOC Project will be essentially completed while the Southeast Plant expansion will not yet be completed. Dry weather flows from the Marina and Beach areas only (15 mgd peak dry weather flow) will enter the NSOC transport-storage facility and will be pumped by the NSPS to the North Point Plant for treatments. Dry weather flow from Jackson Street will continue to be pumped by the Drumm Street Pump Station to the North Point sewer main for gravity flow to the North Point Plant, along with the Sansome Street flow. The Embarcadero leg of the NSOC transport-storage facility will not be in use during dry weather. Figure III-5 shows the schematic for this mode of operation.

FIGURE III-5

STAGE I DRY WEATHER FLOW OPERATION FOR THE
NORTH SHORE OUTFALL CONSOLIDATION PROJECT

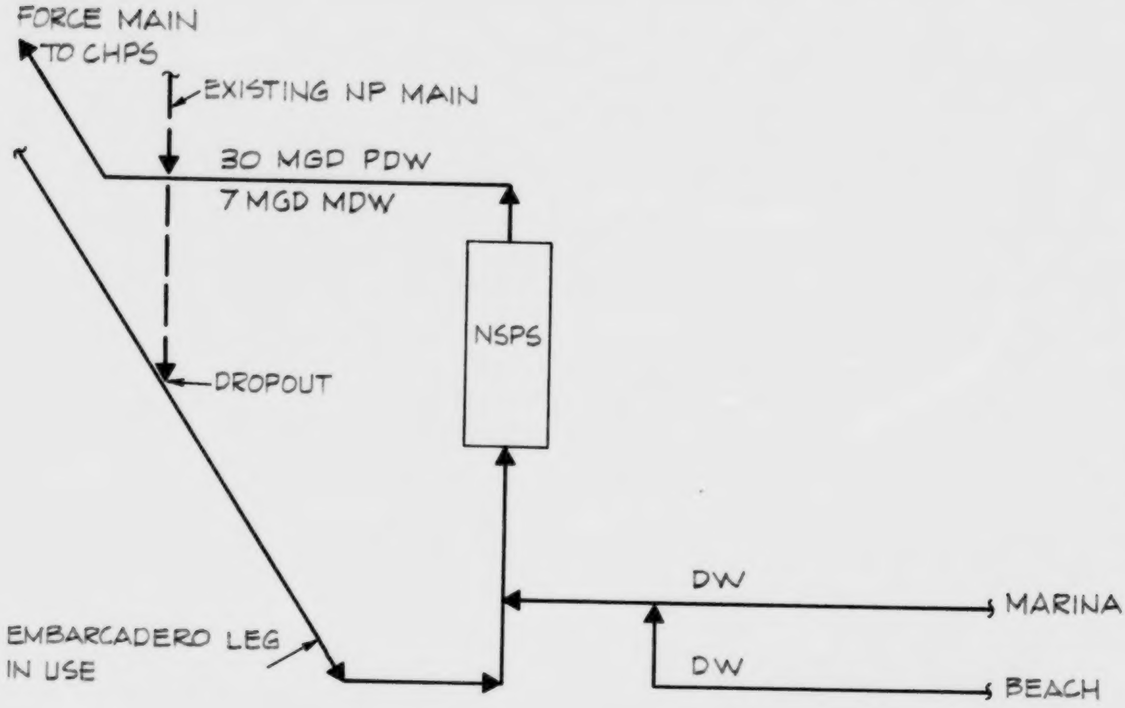

DRUMM ST.
PUMP STA.
EXISTING
NP MAIN
15 MGD PDW
EXISTING
NPWPCP
DW
NSPS
EFFLUENT TO
EXIST. OUTFALL
DW
MARINA
DW
BEACH
EMBARCADERO LEG
NOT IN USE

Stages II and III

At these stages, the expansion of the Southeast Plant will be completed and the plant will be ready to provide secondary treatment for all the dry weather flow from the North Shore area. Figure III-6 depicts the dry weather operations during these stages. The Embarcadero leg will be in use to transport the Jackson Street and Sansome Street flows to the NSPS. All the dry weather flow (30 mgd maximum) will be pumped by the NSPS to the Channel Outfalls Consolidation structure for delivery to the Channel Pump Station (CHPS) for pumping to the Southeast Plant. The North Point Plant will not be used for dry weather treatment.

FIGURE III-6

STAGE II AND III DRY WEATHER FLOW OPERATION FOR THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT


FORCE MAIN TO CHPS
EXISTING NP MAIN
30 MGD PDW
7 MGD MDW
NSPS
DROPOUT
DW
MARINA
DW
BEACH
EMBARCADERO LEG IN USE

Wet Weather Operation

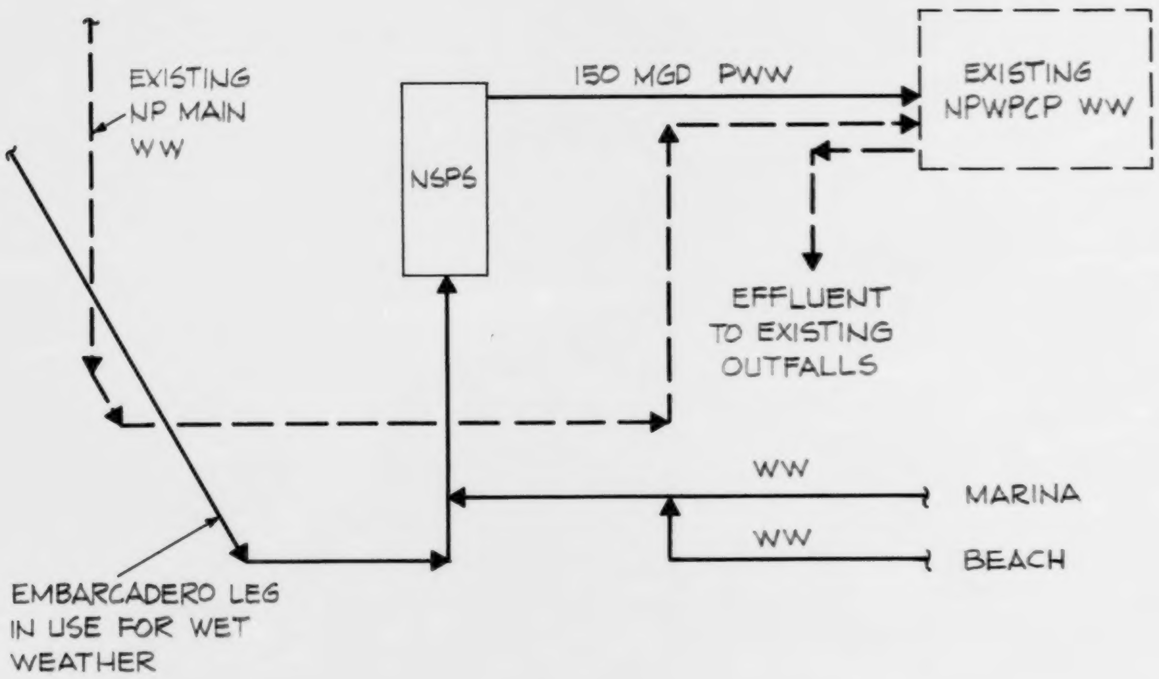
Stage I

As mentioned previously Stage I represents the period when the NSOC Project will be essentially completed while the Southeast Plant will not yet be completed. Figure III-7 graphically illustrates the operations under this stage. Wet weather flows from the Sansome Street area will continue to enter the North Point Plant under gravity flow via the North Point main sewer, while all other flows will be intercepted by the NSOC transport-storage facility and transported to the NSPS to be pumped to the North Point Plant.

The North Point Plant will continue to perform its present function of chemical-primary treatment of up to 150 mgd of wet weather flow. All treated effluent will go to existing outfalls for disposal. Flows that exceed the treatment capacity will be stored in the NSOC transport-storage facility until they can be treated. If storage is exceeded, untreated overflows will occur at selected existing overflow points as presented in the overflow priority table (Table III-2).

FIGURE III-7

STAGE I WET WEATHER FLOW OPERATION FOR THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT

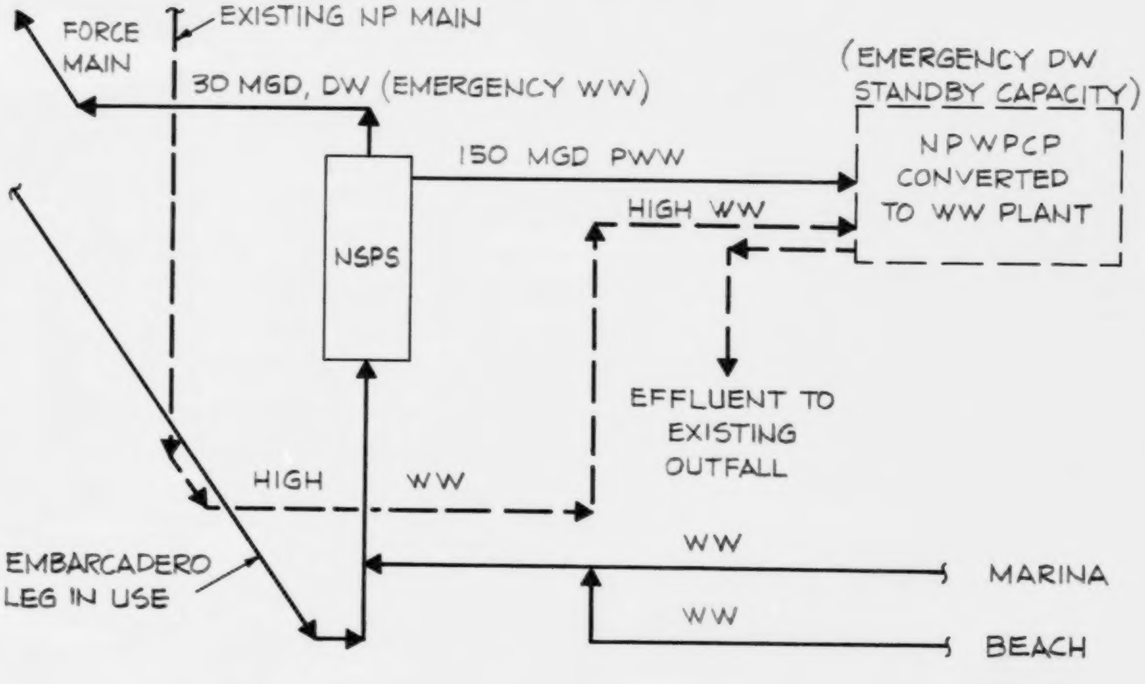

EXISTING NP MAIN WW
150 MGD PWW
EXISTING NPWPCP WW
NSPS
EFFLUENT TO EXISTING OUTFALLS
WW
MARINA
WW
BEACH
EMBARCADERO LEG IN USE FOR WET WEATHER

Stage II

At this stage, the expansion of the Southeast Plant will be completed. The NSPS will function during wet weather exactly as in Stage I, but the North Point Plant will now operate only during wet weather periods. After receiving chemical-primary treatment, the North Point Plant effluent will be discharged through existing outfalls. Figure III-8 presents the schematic for this stage.

FIGURE III-8

STAGE II WET WEATHER FLOW OPERATION FOR THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT



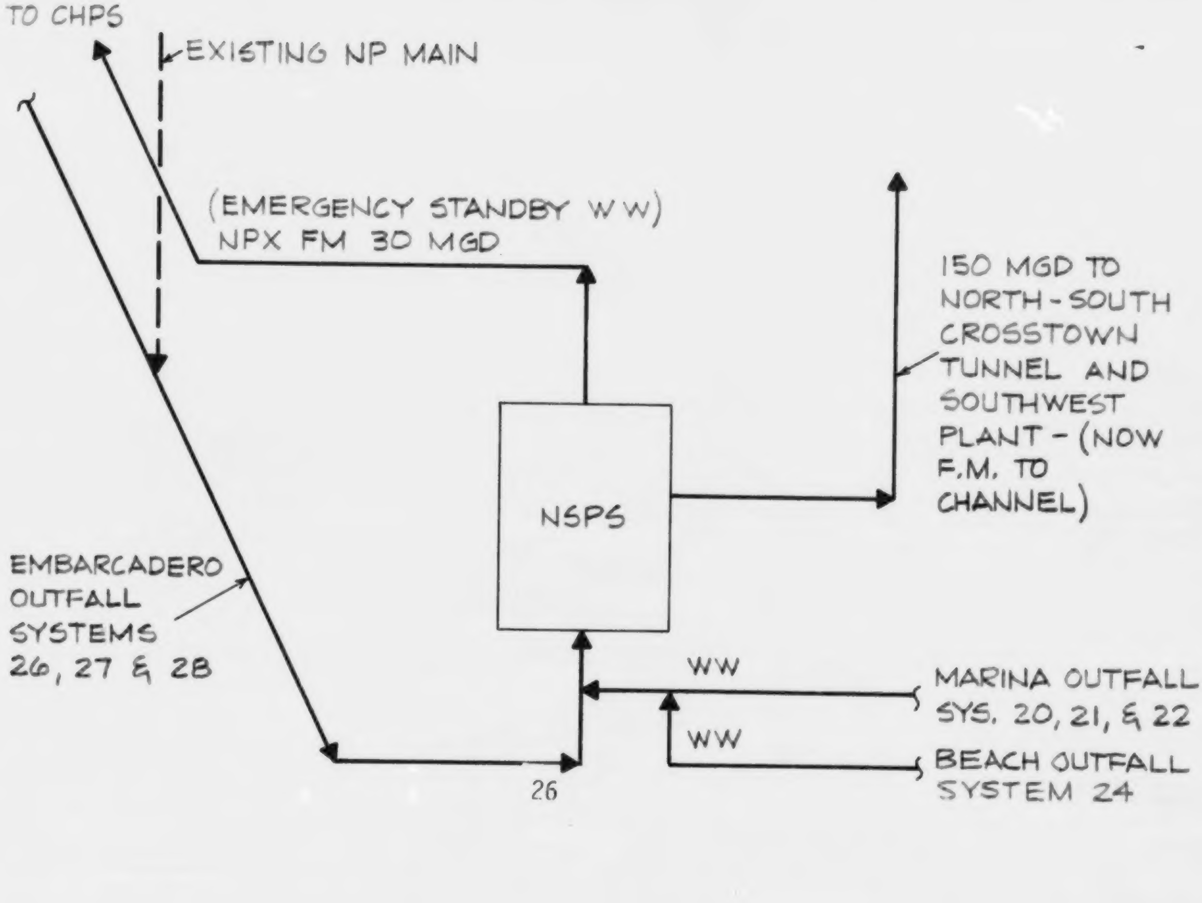
FORCE MAIN
EXISTING NP MAIN
30 MGD, DW (EMERGENCY WW)
(EMERGENCY DW STANDBY CAPACITY)
150 MGD PWW
NPWPCP CONVERTED TO WW PLANT
HIGH WW
NSPS
EFFLUENT TO EXISTING OUTFALL
HIGH
WW
EMBARCADERO LEG IN USE
WW
MARINA
WW
BEACH

Stage III

At this final stage of implementation the Southwest Plant and north-south crosstown tunnel will be completed. Figure III-9 illustrates the planned operations at this stage. All wet weather flows will enter the NSOC transport-storage facility and up to 150 mgd will be pumped by the NSPS to the crosstown tunnel for transport to the Southwest Plant to receive advanced primary treatment and ocean disposal. The North Point Plant and treated effluent outfalls will be completely phased out. Overflows will occur at selected existing overflow points or through two deep water outfalls, should they prove cost-effective.

FIGURE III-9

STAGE III NSOC WET WEATHER FLOW OPERATION FOR THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT


TO CHPS
EXISTING NP MAIN
(EMERGENCY STANDBY WW)
NPX FM 30 MGD
150 MGD TO NORTH-SOUTH CROSSTOWN TUNNEL AND SOUTHWEST PLANT - (NOW F.M. TO CHANNEL)
NSPS
EMBARCADERO OUTFALL SYSTEMS 26, 27 & 28
WW
MARINA OUTFALL SYS. 20, 21, & 22
WW
BEACH OUTFALL SYSTEM 24

GENERAL PERFORMANCE CHARACTERISTICS

The performance of the NSOC Project will be dependent on physical operational constraints, and on operational controls and strategies. These topics are dealt with in later parts of this report, but we feel it will be useful to present here some of the early general findings that characterize the system performance under the planned Basic Operating Strategy. These preliminary findings are important because they serve to point out some of the possible system constraints and to identify control devices that may be needed. Also, they suggest possible control strategies and evaluation criteria.

The following comments apply to the "Basic Operating Strategy" that would minimize overflows by operating the pumping and treatment facilities at the highest possible rates at the earliest possible times. No constraints are considered. Many of these conclusions are based on output from the computer model, TREAT, which was used to analyze the long-term operational performance of the NSOC. These conclusions are:

1. Utilization of storage will be inefficient. Most of the time that storage will be used, a very small percentage of the available storage volume will be filled. This result implies that much more wet weather flow could receive secondary treatment without exceeding the storage capacity.

2. The NSPS and the North Point Plant wet weather pumps will be operated frequently for short periods of time. This finding suggests that these facilities may constrain the control strategies that can be used and the resulting system performance.

3. Rainfall events that are most likely to cause overflows have durations under 6 hours. This finding is particularly significant because it has a direct bearing on the rainfall prediction capability that will be needed if a predictive operating strategy is used.

4. Rainfall events that will most likely cause overflows occur mainly in the period November-March. There is a reduced chance of overflows in October and April, and almost no chance of overflows in the May-September period. These results suggest the use of different operating strategies from month to month.

5. Based on the capacities of existing outfalls and the estimated overflows rates, only four outfalls will be needed for the vast majority of overflow events; for extremely rare events (return periods over 50 years) five outfalls will be needed. These results raise the question as to whether the proposed deep water outfalls will ever prove cost effective.

6. Physical limits of the NSOC Project will result in excess storage capacity for the Marina district in comparison to storage for the Beach and Embarcadero districts. Thus, there will be a need to transfer water from one part of the system to the other to control overflow locations. Preliminary results indicate that an almost level hydraulic grade in the sewer can be maintained.

7. Use of the Baker Street DAF Facility could reduce the number of overflows by about 20 percent, but only if some cost-effective means is found to quickly get the plant on-line and to pump wastewater to it.

CONSTRUCTION SCHEDULE

The NSOC Project is being built under five contracts, N-1 through N-5. The portions of the project included in each contract are identified in Figure III-2. Construction is underway on the N-3, N-4, and N-5 contracts. Design investigations, preliminary designs and specifications have been completed for the two tunnel contracts (N-1 and N-2). Final design and construction on these two contracts was delayed as a result of construction technique and sizing questions. However, these questions have now been resolved and the contracts are out to bid. Construction should begin in mid-1979. The latest estimated construction schedule is shown in Table III-4.

TABLE III-4

PRESENT CONSTRUCTION SCHEDULE FOR THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT

Contract	Description	Estimated Start of Construction*	Estimated Completion of Construction*
N-1	Fort Mason Tunnel	July 1979	January 1982
N-2	North Point Tunnel	August 1979	January 1982
N-3	Embarcadero Leg	June 1977	February 1980
N-4	Marina Leg	May 1978	December 1979
N-5	North Shore Pump Station	October 1977	November 1980

*As of 5/30/79

IV. REAL TIME CONTROL CONCEPTS

NEED FOR CONTROL

Automatic computer control is a concept that is being given increased attention as a means of achieving cost-effectiveness in urban stormwater management. Computer control has been applied with considerable success in industry for more than 15 years. Only recently, however, have a few U.S. cities implemented limited scale computer systems for controlling combined sewer overflows, with a number of other cities in various stages of planning for such systems.

The need to better control combined sewer overflows has become more clear as the impacts of such discharges have been measured and studied. Recent concern over the environmental effects of stormwater and other non-point sources of pollution in 208 studies across the nation has further called attention to the problem. These effects have been particularly significant where the discharges enter water bodies with limited assimilative capacity. In these cases, impacts on beneficial uses can persist for long periods of time after an overflow occurs.

Funding cost-effective solutions to the combined sewer overflow problem is not a straight-forward or simple process. Sewer separation in existing urbanized areas is very costly and often is disruptive; also it fails to address the stormwater pollution problem. Therefore, increased attention has been given to solutions which are directed at giving some measure of treatment to most of the combined flows before they enter receiving waters. The most cost-effective manner of doing this may involve finding the proper tradeoff between the volume and distribution of detention storage and the level and capacity of treatment. The transport, storage, pumping and treatment facilities that are needed will likely form a complex system characterized by many operational variables that can greatly affect the performance of the system. To achieve the best possible performance from a given system, it is important to control these operational variables. Proper control can allow facilities to be sized and operated in the most cost-effective manner possible to meet the desired objectives.

CONTROL SYSTEM OBJECTIVES

The objectives of a control system will vary somewhat depending on such factors as constraints imposed by the existing and proposed facilities, receiving water characteristics, budget constraints, and applicable laws and standards. The objective of the San Francisco NSOC RTAC system are:

1. Limit untreated overflows to an average of less than 1 per year.*
2. Control the location of overflow points.
3. Provide the best treatment possible to all flows.
4. Make optimal use of all facilities to avoid higher costs.
5. Provide data logs and reports for current needs and for historical records.

The first and primary objective is based on a requirement imposed by the California Regional Water Quality Control Board. This requirement was the basis for the design of the NSOC Project. The control system is needed to assure that the requirement can be met with the existing design in a cost-effective manner. When an overflow is about to occur, the system objective becomes one of priority of the location for discharge since certain outfalls are more desirable than others in terms of water quality impacts.

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*The City of San Francisco has recently successfully applied to the California Regional Water Quality Control Board to have the number of allowable overflows increased to 4 per year.

The third objective is to make maximum use of the storage facilities available to provide, to the extent possible, secondary treatment of wet weather flows. When this is not possible, the next best level of treatment available (primary treatment) should be utilized. We note that this objective may be constrained by the first objective; i.e., secondary treatment cannot be given to all wet weather flows without causing increased occurrences of overflows.

The fourth objective is to make optimal use of all facilities by eliminating unnecessary short-term wet weather operation of the North Shore Pump Station, and therefore the North Point Plant. This objective also impacts the number of overflows, since delaying plant operations with the expectation that they will not be required can result in overflows that could have been avoided by prompt plant start-up. The fifth objective includes accumulating data, performing any required calculations, storing data for historical records, and printing out operational logs and reports required by federal and state regulatory agencies.

METHODS OF CONTROL

There are three alternative methods of control that were considered for the NSOC System:

1. Remote manual control with data logging,
2. central digital (automatic) control and data logging, and
3. distributed digital control and data logging.

The above methods actually relate to alternatives available for an overall, City-wide control system. Because this study addresses itself to only the North Shore area, the alternatives available were limited to:

1. Remote manual control with data logging, and
2. digital (automatic) control and data logging.

The reason for this limitation is that in a large centralized system or in a distributed system, the North Shore area would only be one of the areas under control in a City-wide system. As these future systems are not a part of this study, the distinction between the North Shore system being a small central system that will be expanded in the future or a distributed satellite of a larger future distributed system is academic.

In any event, all three alternatives are described for informational purposes to show how the North Shore system would fit into a larger hierarchical system. In the description of each alternative the anticipated staffing for the control center is given for both the future City-wide control system and the proposed North Shore control system. It should be noted that only the staffing for the North Shore control system is considered in the cost comparison (Chapter VII), and that staffing of the North Shore system on a permanent basis will not be required for a City-wide control system.

The first concept, remote manual control with data logging, is illustrated in Figure IV-1. It would provide for a central control facility with a graphic control panel and the computer hardware required for data logging. The control panel would contain the usual indicators, displays, recorders, switches, and controllers. This operational concept would require two operators per shift for the North Shore system, and three for the City-wide system. The operators would review the computer printouts, monitor the control panel, and manually control the system to prevent overflows. At each local control structure there would be facilities for local manual override and control of the system in case of failure.

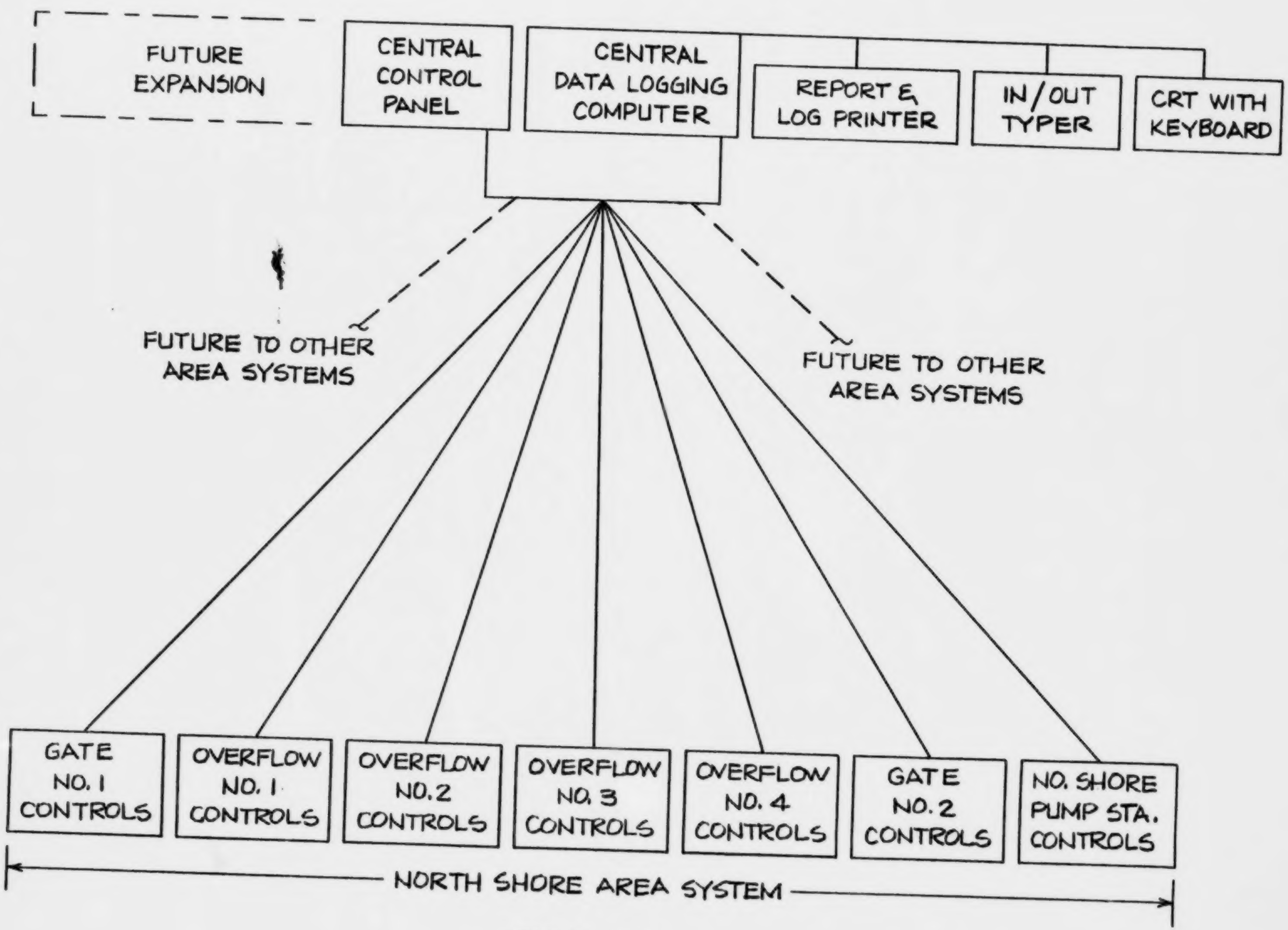

FUTURE EXPANSION
CENTRAL CONTROL PANEL
CENTRAL DATA LOGGING COMPUTER
REPORT & LOG PRINTER
IN/OUT TYPER
CRT WITH KEYBOARD
FUTURE TO OTHER AREA SYSTEMS
FUTURE TO OTHER AREA SYSTEMS
GATE NO. 1 CONTROLS
OVERFLOW NO. 1 CONTROLS
OVERFLOW NO. 2 CONTROLS
OVERFLOW NO. 3 CONTROLS
OVERFLOW NO. 4 CONTROLS
GATE NO. 2 CONTROLS
NO. SHORE PUMP STA. CONTROLS
NORTH SHORE AREA SYSTEM

FIGURE IV-1

REMOTE-MANUAL METHOD OF CONTROL AS RELATED TO THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT

The second concept, central digital control and data logging, is illustrated in Figure IV-2. It would provide for a central control facility with emergency manual back-up panel and computer hardware for control and logging. The computer hardware would include peripherals for observation, logging, and control of the system. The operator's interface with the system would be via a video display (CRT) and keyboard. This operational concept would require one man per shift for the North Shore system, and an average of one and a half men per shift for the City-wide system. These individuals would supervise the automatic control of the system via the computer and would have the capability to take over control of the system if desired.

It should be noted that the system is set up to advise the operator as to the control actions being taken and to allow the operator to abort the control and substitute some other action, if desired. With this central approach for a City-wide system, a dual computer system would be required for reliability as all control would be dependent on the central computer.

The third concept, distributed digital control and logging is shown in Figure IV-3. The central control facility, as described in the second concept, would have supervisory control over a number of area control centers, or satellites, one of which would be provided for the North Shore system. The central computer would perform the hierarchial computing of weather and storm data and would communicate with the satellite computers to provide supervisory control. The satellite computers would control the various elements and facilities in their area based on measurements made within the system. These measurements would include such items as water levels and gate positions. The control objectives of the satellite systems would be directed by the central supervisory computer data and predictions.

The computer hardware at the central control facility would include peripherals for observations, logging, and supervisory control. At the satellites, the computer hardware includes an input-output device such as

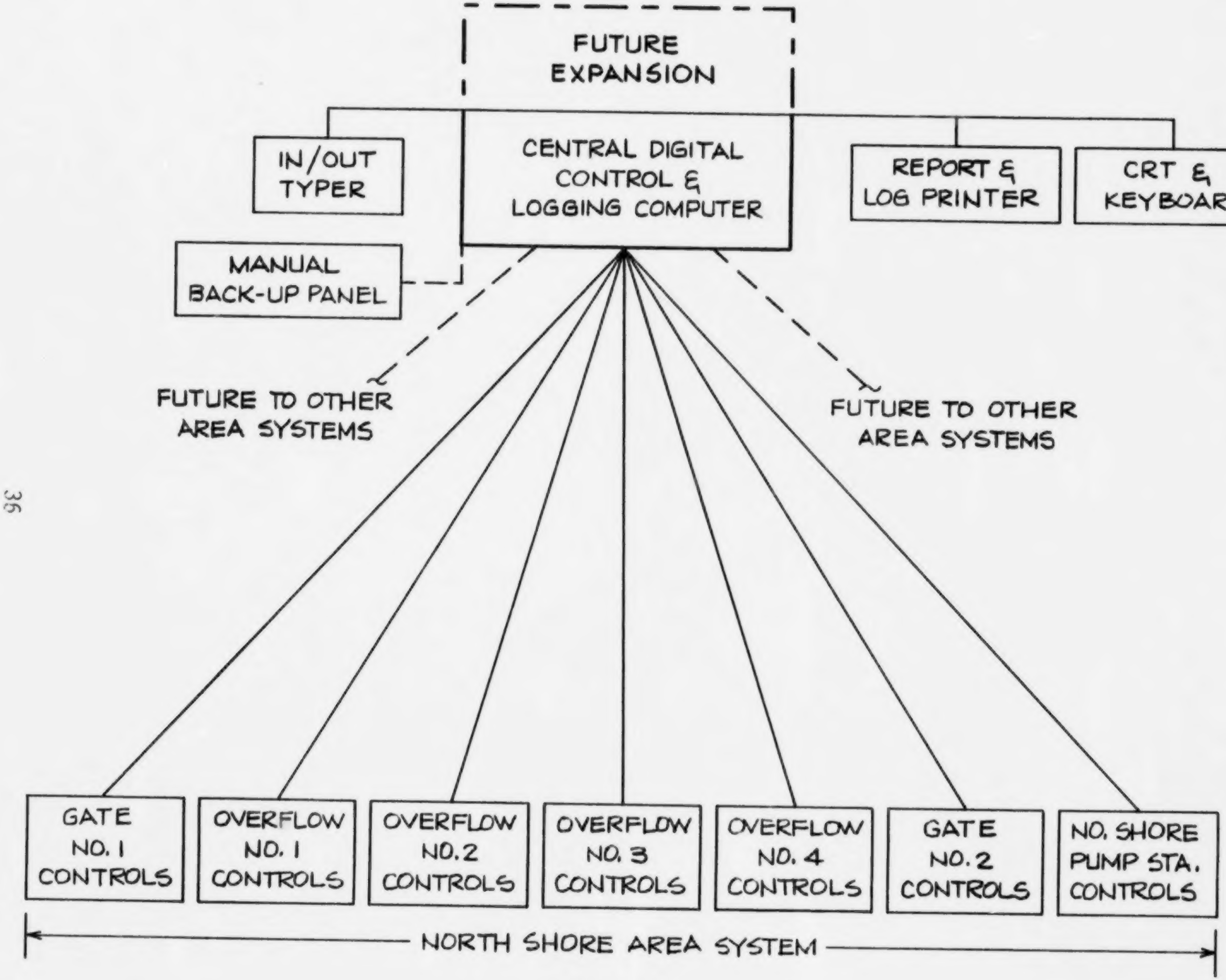

FUTURE
EXPANSION
IN/OUT
TYPER
CENTRAL DIGITAL
CONTROL &
LOGGING COMPUTER
REPORT &
LOG PRINTER
CRT &
KEYBOARD
MANUAL
BACK-UP PANEL
FUTURE TO OTHER
AREA SYSTEMS
FUTURE TO OTHER
AREA SYSTEMS
GATE
NO. 1
CONTROLS
OVERFLOW
NO. 1
CONTROLS
OVERFLOW
NO. 2
CONTROLS
OVERFLOW
NO. 3
CONTROLS
OVERFLOW
NO. 4
CONTROLS
GATE
NO. 2
CONTROLS
NO. SHORE
PUMP STA.
CONTROLS
NORTH SHORE AREA SYSTEM

FIGURE IV-2

CENTRAL DIGITAL CONTROL AND DATA LOGGING AS APPLIED TO THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT

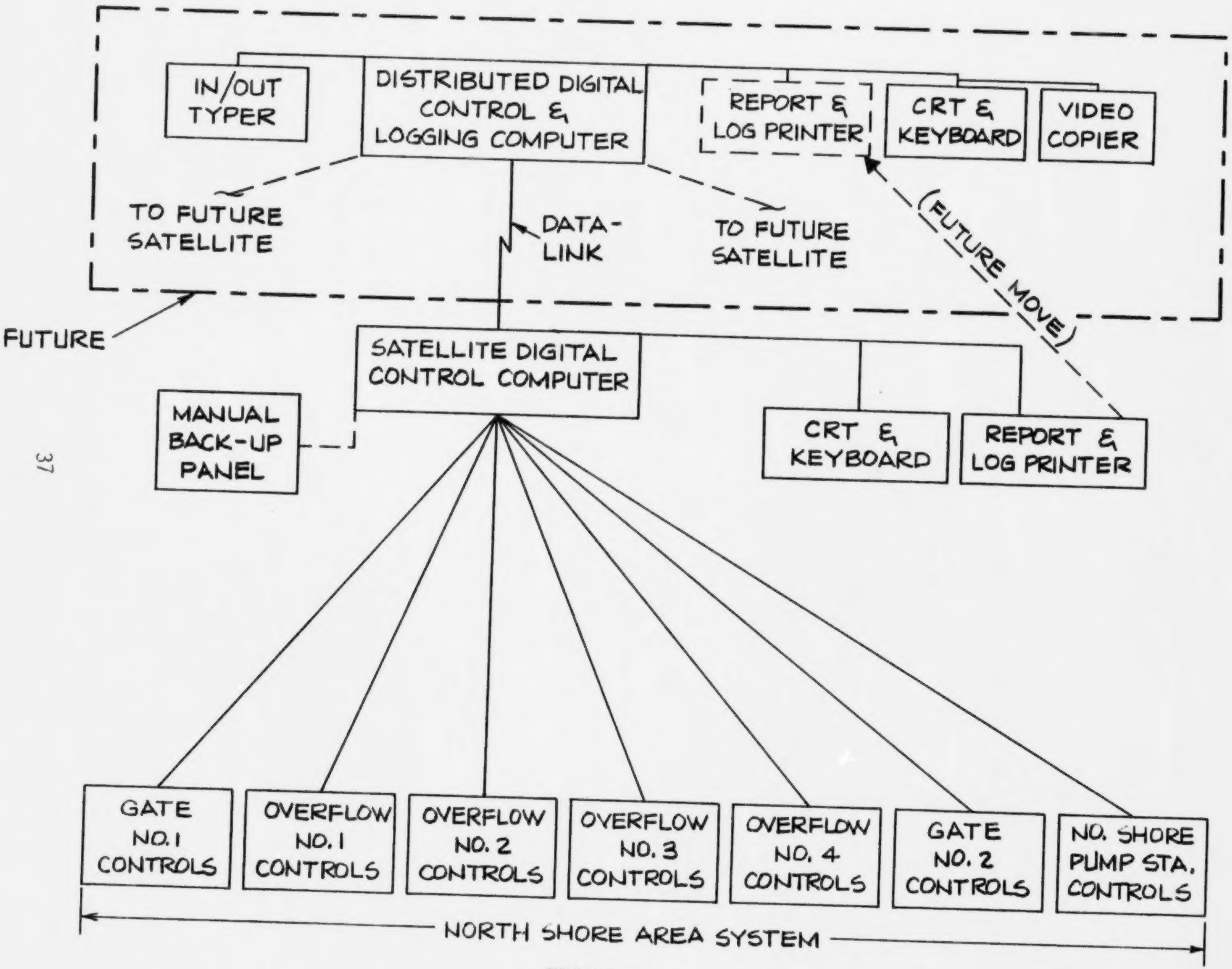

IN/OUT TYPER
DISTRIBUTED DIGITAL CONTROL & LOGGING COMPUTER
REPORT & LOG PRINTER
CRT & KEYBOARD
VIDEO COPIER
TO FUTURE SATELLITE
DATA-LINK
TO FUTURE SATELLITE
(FUTURE MOVE)
FUTURE
SATELLITE DIGITAL CONTROL COMPUTER
MANUAL BACK-UP PANEL
CRT & KEYBOARD
REPORT & LOG PRINTER
GATE NO.1 CONTROLS
OVERFLOW NO. 1 CONTROLS
OVERFLOW NO. 2 CONTROLS
OVERFLOW NO. 3 CONTROLS
OVERFLOW NO. 4 CONTROLS
GATE NO. 2 CONTROLS
NO. SHORE PUMP STA. CONTROLS
NORTH SHORE AREA SYSTEM

FIGURE IV-3

DISTRIBUTION DIGITAL CONTROL AND DATA LOGGING AS APPLIED TO THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT a CRT and keyboard, which is manned in the event of a failure in the central supervisory computer or the communications links or for other reasons. This concept requires one man per shift for the North Shore system and two men per shift for the City-wide system. A dual computer system would not be required for a distributed system as the reliability is provided by the distributed responsibility of the sytem.

CONTROL PHILOSOPHY

Two control philosophies are considered feasible for the NSOC System:

1. Purely reactive control
2. Integrated reactive-predictive control

Five types of control strategies were studied for the North Shore system. They are:

I	Basic Operating Strategy
II	Basic Operating Strategy with Seasonal Constraints
III	Basic Operating Strategy with Minimum Storage Constraints
IV	Basic Operation Strategy with Seasonal and Minimum Storage Constraints
V	Basic Operating Strategy with Rainfall Prediction and Seasonal and Minimum Storage Constraints

Of the above five strategies, the first four are purely reactive control strategies, whereas the fifth strategy provides an integrated, reactive-predictive type of control. The method of control employed to implement any of these strategies may be either manual or automatic. Unless noted to the contrary, it is assumed that the control methods would be automatic with manual override capability. These alternative control strategies are described in detail in Chapter VII.

The reactive control philosophy provides for automatic control based solely on water level, flow, and gate status measurements in the system. In control terminology, this is a feedback control system. The operator, however, would be able to override the automatic control and control the system manually, if desired, based on information and data available at the control facility.

The proposed automatic controls for the four reactive type control strategies are as follows:

I. Basic Operating Strategy - (Common to all five strategies) The control system for the basic operating strategy monitors the volume of water in storage in the system and actuates the No. 1 priority overflow point (Baker Street) when storage reaches or approaches the capacity of the system. The control system would also monitor the discharge flow from the primary overflow point; when the flow reaches capacity and storage remains full, the control system would actuate the No. 2 priority overflow point (Beach Street). The control system would also monitor the discharge from both the Baker Street and Beach Street overflows, and when both flows reach capacity, and the storage still is full, the control system would actuate the No. 3 priority overflow point (Sansome Street). With three overflows open and the control system monitoring the flow from each, if all are at capacity and storage is still full, the control system would actuate the No. 4 priority overflow point (Jackson Street) and monitor its discharge.

Once an overflow event has occurred, the control system would monitor storage volume and compare discharge to the discharge capacity at each overflow point. It would then act to close any unnecessary overflow points starting with the lowest priority point, or the last overflow point to be opened. This procedure would continue until only the No. 1 priority overflow point (Baker Street) remains open. This point would be closed once the volume in storage is less than capacity. After an overflow event, the control system would determine that all overflow points are closed; the control system would then go into a flushing mode and monitor the volume in storage. When the volume diminishes to a pre-set value, a flushing gate would be closed until the level behind it reaches a preselected elevation, at which time the gate would be opened and the water behind would flush the sewer. The flushing gates would be sequenced, with time allotted between them, so the North Shore Pump Station would not be overburdened.

II. Basic Operating Strategy with Seasonal Constraints

The control system for this strategy operates the overflow points and flushing gates in the same manner as the basic operating strategy. In addition, the control system for this strategy would provide a means for interlocking the two wet weather pumps at the North Shore Pump Station to prevent them from automatically starting during preselected months of the year. When the pumps are allowed to run, they would be under the control of the North Shore Pump Station control system.

III. Basic Operating Strategy with Minimum Storage Constraints

The control system for this strategy would operate the overflow points and flushing gates in the same manner as the basic operating strategy. In addition, the control system for this strategy would provide a means for interlocking the two wet weather pumps at the North Shore Pump Station to prevent them from automatically starting until the control system determines that a preset minimum volume of storage has been attained within the storage system. Once the pumps are allowed to start, they would be under the control of the North Shore Pump Station control system.

IV. Basic Operating Strategy with Seasonal and Minimum Storage Constraints

The control system for this strategy is a combination of the first three strategies. It operates the overflow points and flushing gates in the same manner as the operating strategy, while limiting the allowable operating months of the wet weather pumps as in Strategy II. When pump station operation is allowed, it would assure that a preset volume in storage is attained before permitting the pumps to start, as in Strategy III, except that this preset minimum volume in storage is different for each month.

The integrated reactive-predictive control philosophy provides for automatic control based on flow level and gate status measurements in the system as well as predictions derived from either real or simulated rainfall and storm runoff and transport modeling. The control system would operate in a reactive mode with predictive adjustments. In control terminology, this is a feedforward control system with feedback trim or compensation. If desired, it would also be possible to override the predictive aspect of the control or for the operator to take over control manually, utilizing any or all of the data available at the control facility.

The proposed control system for the fifth strategy utilizes a reactive-predictive philosophy with the reactive component being essentially the same as Strategy IV, above. Since the NSOC System responds quickly to rainfall due to its size and urban character, remote raingage data is required on which to base predictions with a sufficient lead time to be useful. A rainfall prediction model is needed to relate expected future rainfall in the NSOC area to rainfall occurring at remote areas. Chapter VI describes a model developed for this RTAC Study. The predicted rainfall would then be converted to predicted runoff and storage levels by runoff and transport computer models such as those described in Chapter V. These storage level predictions would then be used to modify the reactive rules that control the operation of the wet weather pumps or other control devices. It should be noted that the computing power required, hardware and software, is substantially increased due to the necessity of running simulation models in a real time mode for prediction.

LOCATION AND TYPES OF CONTROL DEVICES

Independent of the control method and philosophy utilized, local control devices and monitors will be required for the NSOC System to operate.

To meet the objectives of the control system, devices within the physical transport, storage, and treatment facilities will be needed to control gates, overflow weirs, inflatable dams, water level monitors, and flow monitors. Gates would serve to control the location of storage in the system, if desired, and thus maximize the use of storage. They would also provide the capability to flush the system after a storage period and thus reduce the buildup of settled solids. Overflow weirs and inflatable dams would provide control at overflow points and permit control of the location of overflows. Water level and flow monitors would provide information on the current status of the system. This information is needed for making control decisions and for reports and historical records.

To illustrate the control devices that would be needed in the NSOC System, a schematic of the operational features of the system as designed is shown in Figure IV-4. At this time, it is felt that the additional features shown in the revised schematic in Figure IV-5 will be sufficient to meet the control objectives. The additional control devices needed and their locations are the same for all proposed control strategies.

Flushing gates are shown at two locations in the NSOC sewer, just downstream of the intersection of Pierce Street and Marina Boulevard (at the tunnel portal) and just downstream of Sansome Street (gates D and E in Figure IV-5). The existing gates at the North Shore Pump Station (gates A and B) will not be under the direct control of the RTAC system. However, the position of all gates will be monitored and transmitted back to the control center.

The overflow points selected, in order of desirability, are the Baker Street outfall, the Beach Street outfall, the Sansome Street outfall and the Jackson Street outfall. Therefore, each of these locations should be controlled by either a fixed height overflow weir, an inflatable dam, or a control gate. A combination of a weir and an inflatable dam at each overflow would probably be best. The weir could be set at the best elevation for flow monitoring while the dam would effectively block-off the overflow until such time that overflow at that point was required. All other overflow points in the system should be blocked-off once sufficient time has elapsed to determine that they will no longer be needed as projected.

Water levels should be monitored upstream and downstream of each of the flushing gates, at each of the four priority overflow points, at the intake to the Pump Station and possibly at one or more points to monitor tide levels in the Bay.

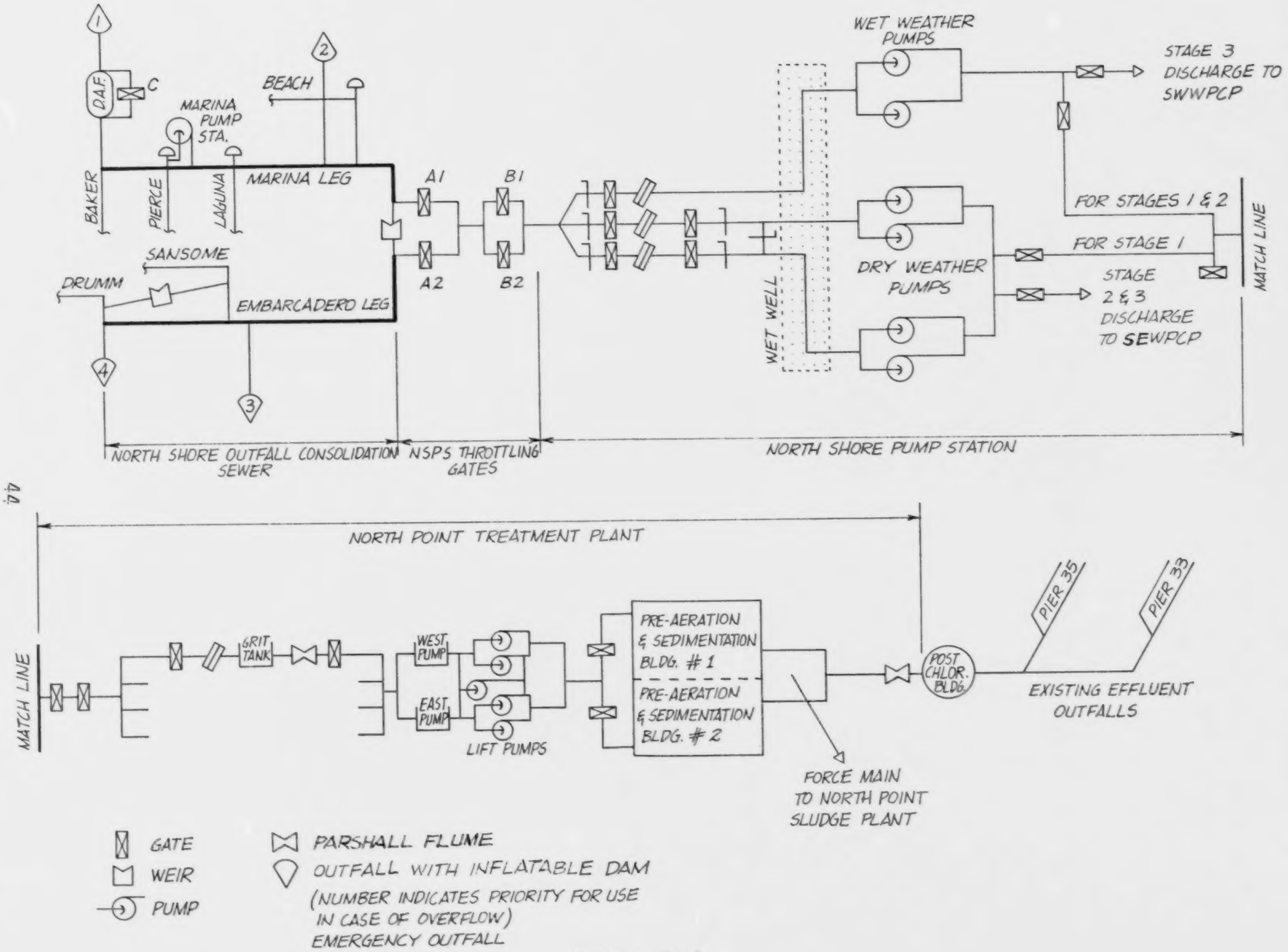
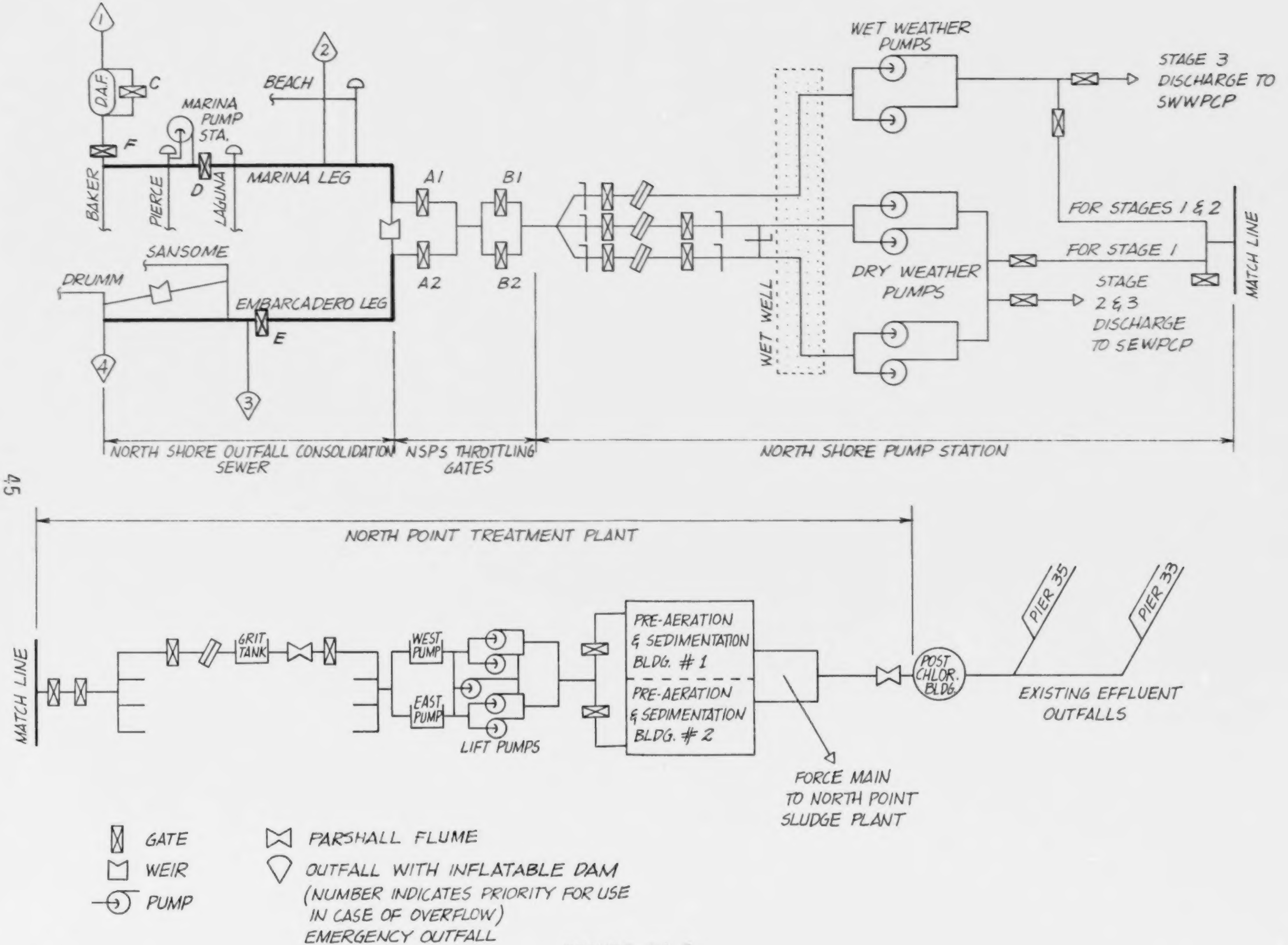

1
D.A.F.
C
2
BEACH
MARINA PUMP STA.
BAKER
PIERCE
LAGUNA
MARINA LEG
SANSOME
DRUMM
EMBARCADERO LEG
4
3
A1
A2
B1
B2
WET WELL
WET WEATHER PUMPS
DRY WEATHER PUMPS
STAGE 3 DISCHARGE TO SWWPCP
FOR STAGES 1 & 2
FOR STAGE 1
STAGE 2 & 3 DISCHARGE TO SEWPCP
MATCH LINE
NORTH SHORE OUTFALL CONSOLIDATION SEWER
NSPS THROTTLING GATES
NORTH SHORE PUMP STATION
NORTH POINT TREATMENT PLANT
MATCH LINE
GRIT TANK
WEST PUMP
EAST PUMP
LIFT PUMPS
PRE-AERATION & SEDIMENTATION BLDG. # 1
PRE-AERATION & SEDIMENTATION BLDG. # 2
FORCE MAIN TO NORTH POINT SLUDGE PLANT
POST CHLOR. BLDG.
PIER 35
PIER 33
EXISTING EFFLUENT OUTFALLS
GATE
WEIR
PUMP
PARSHALL FLUME
OUTFALL WITH INFLATABLE DAM
(NUMBER INDICATES PRIORITY FOR USE IN CASE OF OVERFLOW)
EMERGENCY OUTFALL

FIGURE IV-4

CONTROL FEATURES PRESENTLY DESIGNED IN NORTH SHORE OUTFALL CONSOLIDATION PROJECT



1
2
3
4
D.A.F.
C
F
MARINA PUMP STA.
BEACH
BAKER
PIERCE
D
LAGUNA
MARINA LEG
SANSOME
DRUMM
EMBARCADERO LEG
E
A1
A2
B1
B2
WET WELL
WET WEATHER PUMPS
DRY WEATHER PUMPS
STAGE 3 DISCHARGE TO SWWPCP
FOR STAGES 1 & 2
FOR STAGE 1
STAGE 2 & 3 DISCHARGE TO SEWPCP
MATCH LINE
NORTH SHORE OUTFALL CONSOLIDATION SEWER
NSPS THROTTLING GATES
NORTH SHORE PUMP STATION
NORTH POINT TREATMENT PLANT
MATCH LINE
GRIT TANK
WEST PUMP
EAST PUMP
LIFT PUMPS
PRE-AERATION & SEDIMENTATION BLDG. # 1
PRE-AERATION & SEDIMENTATION BLDG. # 2
FORCE MAIN TO NORTH POINT SLUDGE PLANT
POST CHLOR. BLDG.
PIER 35
PIER 33
EXISTING EFFLUENT OUTFALLS
GATE
WEIR
PUMP
PARSHALL FLUME
OUTFALL WITH INFLATABLE DAM
(NUMBER INDICATES PRIORITY FOR USE IN CASE OF OVERFLOW)
EMERGENCY OUTFALL

FIGURE IV-5

DESIRABLE ADDED CONTROL FEATURES FOR THE NORTH SHORE OUTFALL CONSOLIDATION PROJECT

In addition to these control devices, pumps and gates in other parts of the NSOC System must be controlled. The pumps and gates in the North Shore Pump Station will operate automatically based on the water level in the wet well via the controls provided with this station, with the exception of pump overrides by the RTAC System when required. The start-up of the North Point Plant must be controlled during Stage II (See Chapter III). It has been assumed for the cost comparison that the North Point Plant will be staffed for operation during the required treatment season and that the Plant will be controlled via devices already existing at the Plant. The RTAC System will send alarms to the Plant when plant usage is anticipated and when the wet weather pumps actually start. It was further assumed that the Baker Street DAF Plant would not be used for treatment but only as an untreated overflow point.

The computer hardware and software is somewhat dependent on the method and philosophy of control that is selected. In general, a central processor and related software would be required, along with a color cathode ray tube (CRT) for displays, keyboard for operator interface, a log and report printer, magnetic storage for historical record keeping, a data link for future expansion to a distributed or larger central control system, and a manual operator panel which would either be a limited emergency back-up panel to the automatic control or a full scale manual control panel depending on the method of control selected. Figures IV-6, IV-7 and IV-8 show examples of some of the required hardware.

The best location for this control equipment would be at the North Shore Pump Station Control Room. Utilizing this location allows the control equipment to be at the heart of the North Shore sewer system, which it controls, and provides a convenient point if future expansion leads to City-wide distributed or centralized control. If a City-wide control system is implemented, the location for all control equipment would have to be reviewed.

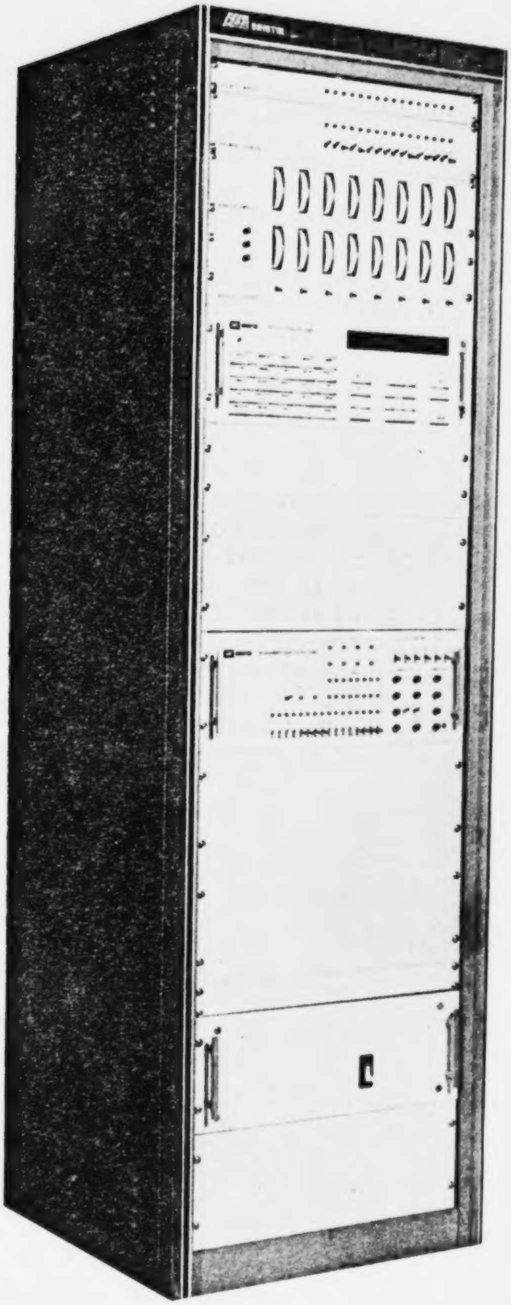


FIGURE IV-6

TYPICAL MICROPROCESSOR BASED CONTROLLER
WITH MANUAL BACKUP

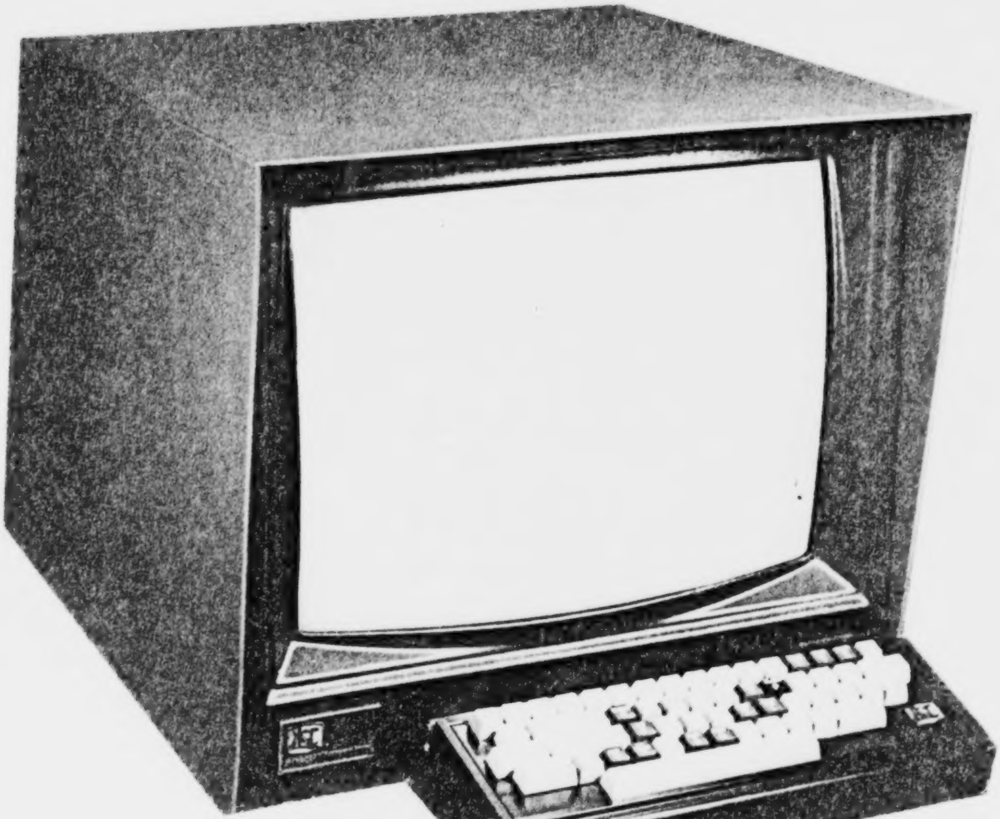


FIGURE IV-7

TYPICAL CRT WITH KEYBOARD

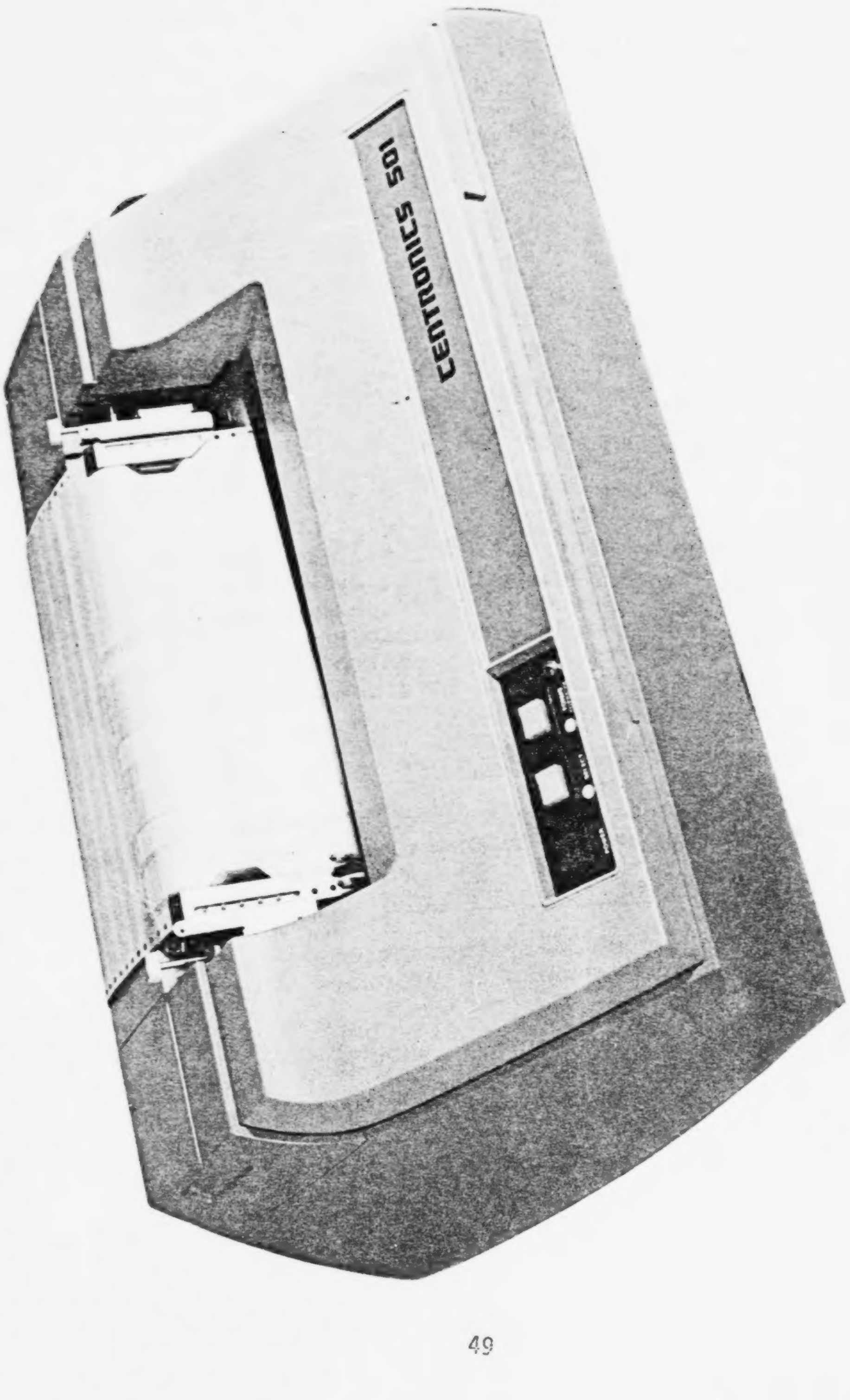

CENTRONICS 501

FIGURE IV-8

TYPICAL PRINTER FOR LOGGING

V. MATHEMATICAL MODELING AND REAL TIME CONTROL

INTRODUCTION

Three computer simulation models were involved in the study of the automatic control systems for the San Francisco NSOC Project; these models are TREAT, the RUNOFF Block of SWMM and the TRANSPORT Block of SWMM. The TREAT model was used to determine information on overflow frequencies and other operational performance measures under several alternative control strategies. The RUNOFF Block of the SWMM model was considered for developing design hydrographs from the subareas that drain into the NSOC system. The NSOC system itself (including the transport-storage facilities, pumps, gates and overflow weirs) was modeled using the TRANSPORT Block of SWMM.

Because these or similar models may be useful in other real time control studies, we have elected to present a brief discussion on how the models were used in this study. For each of the three models, the following will be discussed:

- General description and theory,
- use of the model in RTAC studies, and
- use of the model in the NSOC study.

TREAT MODEL

General Description and Theory

TREAT is a forerunner of STORM, a widely used U.S. Corps of Engineers storm runoff model. Although the two models have very different input-output formats and capabilities,* the basic theory described here applies to both models.

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*STORM has additional optional capabilities, such as unit hydrograph and runoff quality routines, which are not in TREAT.

TREAT converts hourly rainfall over a watershed into hourly runoff using a simple runoff coefficient relation. Time-varying sanitary flows into a combined sewer can also be specified. The combined flows receive treatment at a specified constant rate. If the treatment capacity is exceeded, the excess goes into storage. If the specified fixed available storage volume is exceeded, an overflow occurs (i.e., an untreated discharge). The water in storage that does not overflow receives treatment as treatment capacity becomes available. Figure V-1 illustrates these processes.

Since the computations required by TREAT are relatively simple, it is possible to economically simulate long periods of time (several years). Routines in the program are set up to statistically analyze the results, which include such system performance indicators as the number, amount, and duration of treatment, storage, and overflow events. Because of these capabilities, TREAT is ideal for facility sizing studies which involve trade-offs between storage capacity and treatment rate. If sufficiently long rainfall records exist (or can be generated), the program supplies the necessary information to evaluate the proposed facilities in a probabilistic manner.

While TREAT is a valuable planning tool, its simplistic approach leads to several limitations. The following points must be considered in applying the model and interpreting the results:

- The modified rational method used to convert rainfall to runoff requires that a constant runoff coefficient be applied to each hour's precipitations. Hydrologic processes not included in the model will cause some simulated flows to be too high and others too low. The errors, however, are smaller when most of the runoff is from impervious surfaces, as in urban watersheds.

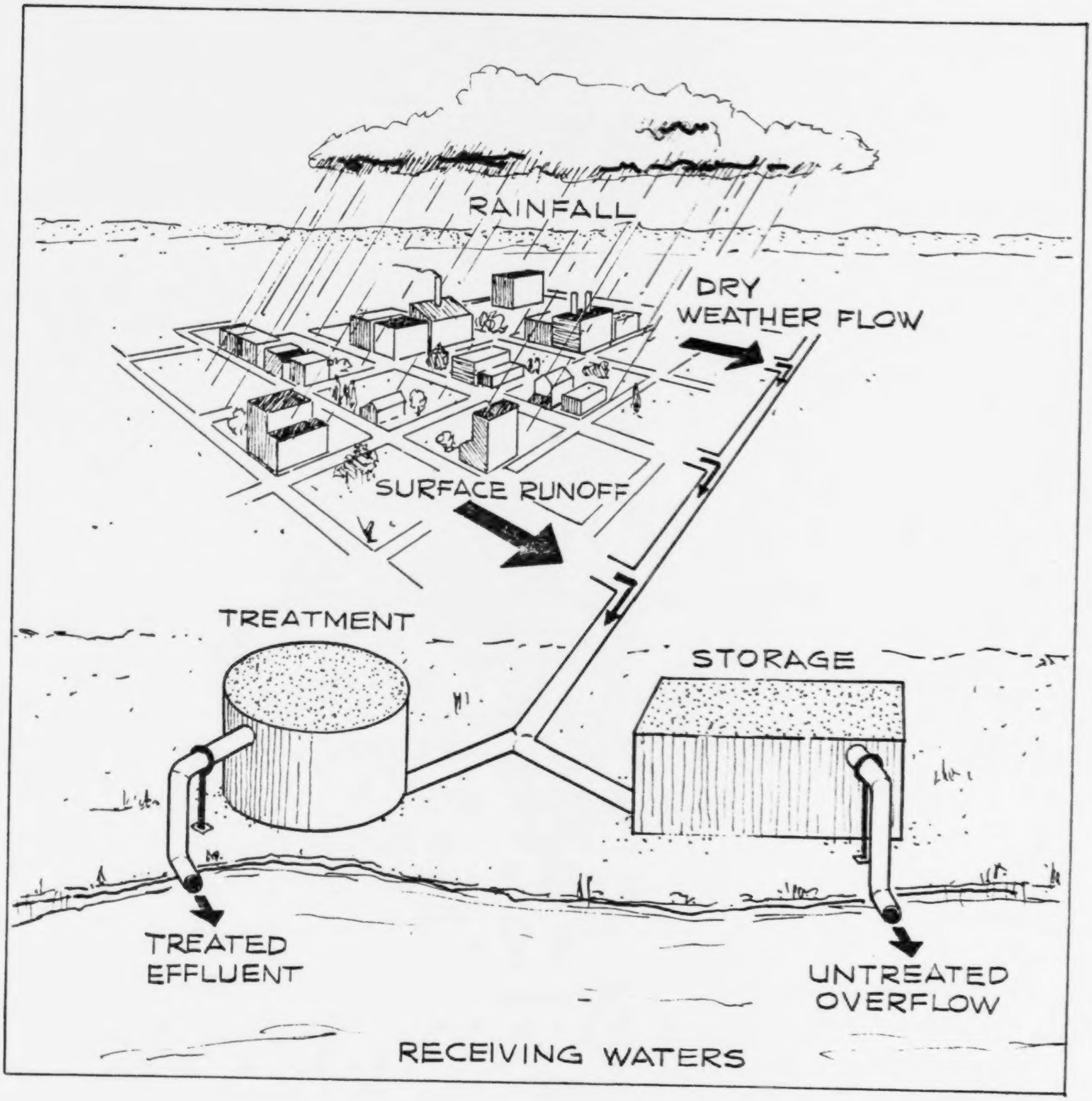
RAINFALL
DRY WEATHER FLOW
SURFACE RUNOFF
TREATMENT
STORAGE
TREATED EFFLUENT
UNTREATED OVERFLOW
RECEIVING WATERS

FIGURE V-1
MAJOR PROCESSES SIMULATION BY THE COMPUTER PROGRAM TREAT

- Since there is no routing in the program, the flows are assumed to occur within the same hour as the rainfall. Therefore, the size of the watershed should be limited to an area having a time of concentration less than the basic time step of the simulation, which is one hour. Larger areas may be used at the cost of accuracy.

- Since the program operates on a time step of one hour, it cannot simulate the response of a watershed or physical facilities to rainfall durations of less than one hour. Therefore, the peak flows predicted by TREAT may be low even if the total volume of flow is correct, especially if the watershed is small and the critical storm duration is less than one hour. Likewise, the use of storage facilities for less than one hour cannot be modeled, so that TREAT may also underestimate the use of storage and the occurrence of overflows having durations under an hour.

Use Of TREAT In RTAC Studies

The capabilities and simplicity of TREAT make it an appropriate and valuable tool in RTAC studies. The hydrologic response of urbanized watersheds tributary to combined sewers and interceptors can be reasonably well simulated by the use of runoff coefficients and one hour time steps. Long-term hourly rainfall data is often available for such areas. The trade-off between storage capacity and treatment rate and its relation to overflow frequency is central to the design and performance of facilities that can profit by the use of automatic control.

Although TREAT cannot model detailed system response to specific control devices, it can economically process long periods of rainfall records, outputing a frequency analysis of system performance indicators.

This information is very useful for characterizing general system response and for developing and evaluating alternative control strategies.

Making initial TREAT runs with a simple basic operating rule can be used to characterize the system performance by answering such questions as: What types of storms cause overflows and when do they usually occur? How often are the treatment facilities needed and for how long? What is the effect of additional treatment or storage capacity? Is the storage capacity properly distributed or will transfers be needed to optimize the use of storage? How many outfalls will be needed?, how often?, and for how long? From the answers to these questions, potential constraints and possible areas for improvement can be identified, leading to the development of alternatives for controlling the system.

Some of the types of control strategies that can be evaluated are related to when the wet weather treatment should be started (whether based on reactive (water levels) or predictive (rainfall forecasts) strategies), and what portion of the year wet weather treatment should be available in areas with distinct dry and wet seasons. The impacts of these strategies on performance indicators such as the number, amount and duration of treatment, storage and overflow events can provide the basis for the evaluation of the alternatives.

Use of TREAT in the NSOC Study

TREAT was used extensively in the NSOC RTAC Study to characterize system performance and to develop and evaluate alternative control strategies. The rainfall data used was the 70-year (1907-1977) record of hourly rainfall values measured at the Federal Office Building (FOB) in downtown San Francisco. Correlation of FOB rainfall with gages in the NSOC project drainage area showed that rainfall values in the Marina district are 1.16 times greater than FOB and values in the Beach and Embarcadero districts are 1.10 times greater than FOB. For the North Shore drainage area as a whole, rainfall is estimated to be 1.12 times greater than FOB values. This rainfall adjustment was made and an initial series of TREAT runs were produced. Several of the conclusions listed under "General Performance Characteristics" in Chapter III were based on the results of these runs. A number of alternative control strategies were then developed and evaluated in a series of TREAT runs, as documented in Chapter VII.

Since the results of the TREAT runs used to evaluate the control strategies are presented in detail in Chapter VII, the results are not presented here.

In short, TREAT was found to be extremely valuable in evaluating alternative control strategies. Its applicability to actual real time control systems is, however, limited. Under real time control, models will only be needed if a reactive-predictive or wholly predictive strategy is selected. In that event, decisions will have to be made more frequently than hourly intervals and the system will have to be modeled with more detail than TREAT permits. *Consequently, TREAT does not appear to be the appropriate model for implementing predictive or reactive-predictive strategies for real time control of combined sewer systems.*

RUNOFF BLOCK - SWMM

General Description and Theory

The RUNOFF Block of the EPA Stormwater Management Model (SWMM) is a computer model which simulates the hydrologic response of a watershed and drainage system to a single storm event. The output from the model is a series of runoff hydrographs at points throughout the drainage system.

The RUNOFF Block of SWMM is applied by first dividing a watershed into a number of smaller drainage areas (subareas). The physical data required for each subarea includes area, slope, hydraulic roughness, depression storage, infiltration, land use, and imperviousness. Flow through gutters, pipes and channels is also simulated, so their dimensions, slopes and roughness coefficients are also needed. The rainfall hyetograph for a storm is input in short time intervals, usually hourly or shorter. The model works on a user specified time step appropriate for the size of the area and the accuracy desired (usually a few minutes). The program routes the rainfall excess (rainfall minus computed depression storage and infiltration) over the pervious and impervious areas and through gutters, pipes, and channels of the drainage system, utilizing uniform flow equations. Hydrographs are thus produced at a number of user-specified locations in the drainage system. If flow measurements of historical storms are available, the model can be calibrated to closely duplicate actual flows. The model also has the capability to simulate the entry and transport of pollutants in the stormwater, although not with the same level of accuracy.

The level of detail of the RUNOFF Block can be varied by user decisions as to size of subareas and length of time steps. In all cases, however, it is hydrologically more detailed than the TREAT model, which assumes a fixed runoff coefficient and has no routing capabilities. It is also much more costly to set-up and execute and therefore is used for single event simulation only. The model is frequently used for both planning and design level studies. The runoff hydrographs produced by the RUNOFF Block may also serve as input to the TRANSPORT Block of SWMM, which further routes the flows through larger drainage facilities with greater detail. Further documentation may be found in "Storm Water Management Model User's Manual, Version II," EPA-670/2-75-017, March 1975, and in San Francisco Stormwater Model User's Manual and Program Documentation, prepared for The City and County of San Francisco, by Water Resources Engineers.

Use of the RUNOFF Block - SWMM in RTAC Studies

An important aspect of developing an RTAC Strategy is understanding how the system will respond hydraulically to wet weather flows under various operating rules. The TRANSPORT Block of SWMM possesses the level of detail and capabilities to model the hydraulic response of a complex controlled system. Since it is a routing model with no runoff computing capabilities, it requires that inflows to the system be supplied as inputs. The runoff hydrographs may be generated by a number of means, such as runoff coefficient, unit hydrograph or computer modeling approaches. Since the RUNOFF Block is directly compatible with the TRANSPORT Block, it provides a convenient and adequate means of generating runoff hydrographs to be routed through the RTAC system. This approach may be especially appropriate for larger drainage areas that include a substantial amount of pervious surfaces, since infiltration and routing effects are significant in such cases. *Also, due to its efficiency, stability, and reliability, the RUNOFF Block could be adapted and would be well suited to real-time control where rainfall and runoff predictions are incorporated into the control decision process.*

Use of the RUNOFF Block - SWMM in the NSOC Study

Because of several factors specific to the NSOC Study, it was not necessary to apply the RUNOFF Block to develop inflow hydrographs to the NSOC sewer. These factors are:

1. The individual subareas draining to the NSOC sewer are relatively small and highly urbanized, minimizing the effects of infiltration and routing.

2. Detailed rainfall data throughout the area was available in 5-minute increments of time.

3. Analysis showed that a simple runoff coefficient approach would be adequate to develop subarea hydrographs in this situation.

These conditions may not exist in other study areas. In those cases RUNOFF or a similar type of model would be an appropriate tool for use in predictive or reactive-predictive control systems to 1) convert predicted rainfall into expected runoff, or 2) estimate runoff from measured rainfall. The latter option should be considered when the time of concentration of the tributary watershed is greater than one hour.

TRANSPORT BLOCK - SWMM

General Description and Theory

The TRANSPORT Block of SWMM dynamically routes runoff hydrographs and time-varying sanitary flows through a major storm drain or sewer system. It produces continuous head curves and hydrographs at selected points in the transport system for a designated storm period. The model can simulate the following elements of the system: pipes, manholes, weirs, orifices, pumps, on-line storage, and outfall structures. Such effects as backwater, tides, surcharges or pressure flow, flow reversals, branching and looping are considered.

The TRANSPORT Block uses a link-node description of the sewer system which facilitates the discrete representation of the physical prototype and the mathematical solution of the gradually-varied unsteady flow equations (Saint-Venant equations) which form the mathematical basis of the model. The flow and continuity equations are solved sequentially for each link and node over a time step. The numerical integration of the finite difference representation of the flow equations is accomplished by a modified Euler method, yielding a completely explicit solution. Although explicit methods are efficient, they may lead to unstable solutions if the time step is too short. The proper time step depends on the shortest, smallest pipe in the network having high inflows; time steps from 10 to 30 seconds are generally acceptable.

Because of the short time steps required and the complexity of the model, it is much more expensive to use than the RUNOFF Block. Therefore, the TRANSPORT Block is normally applied only to those portions of the total system than cannot be modeled by the RUNOFF Block, either because the uniform flow assumption is invalid, or because flow control structures (weirs, orifices, etc.), pumping or storage facilities must be included. Further documentation may be found in "Storm Water Management Model User's Manual, Version II," EPA-670/2-75-017, March, 1975 and in San Francisco Stormwater Model User's Manual and Program Documentation, prepared for The City and County of San Francisco, by Water Resources Engineers.

Use of the TRANSPORT Block - SWMM in RTAC Studies

Two major uses for the TRANSPORT Block in RTAC Studies are:

1. Hydraulic analysis, and

2. Real time simulation.

Because of its capability to model flow control devices and pumping and storage facilities, the TRANSPORT Block of SWMM is a potentially useful tool in assessing the hydraulic response of a complex storm or a combined sewer system design under historical or design storm conditions. The model can simulate, with minor program changes in some cases, the effects of control strategies involving pumping rules, weir overflows and transfers, and other control devices that would be part of an RTAC system. Such simulations may point out potential problems with the prototype response that can be corrected by design changes or by implementation of specific control measures. The model also could be used to formulate and test operating rules for the system and to locate the control devices necessary to meet the control objectives.

The simulations could also indicate whether the assumptions and conclusions made in analyzing the system performance using simpler models (such as TREAT) are reasonable.

The second use of the TRANSPORT Block would be as a real time simulation model which could be used to simulate, in advance, system response to runoff that would occur from rainfall presently falling or from predicted rainfall. In order to do this, it would be necessary to simplify the model for the fast execution and to minimize computer storage requirments. This could be accomplished by simulating only the key elements of the drainage system that must be modeled by TRANSPORT due to backwater effects or control devices. The model could be specifically written for the system and all the physical data hardwired. The computer storage requirements for data output, which are large in the more general TRANSPORT model, could be changed and reduced for real time simulation. Two key questions with respect to the use of this model on a specific system that have to be addressed are:

1. Is a model of this degree of sophistication required for real time simulation?, and

2. Are stability problems in the TRANSPORT Block apt to cause problems in the simulation under real time conditions?

Our experience gained in applying TRANSPORT to the NSOC Project, as described in the following section, has provided us with partial answers to these two key questions.

Use Of The TRANSPORT Block - SWMM In The NSOC Study

Approach

Two key concerns regarding the NSOC Project that required analysis using the TRANSPORT Block were:

1. Would the NSOC transport-storage facility fill up uniformly during a storm as TREAT assumes it does, or would one section of the facility fill up more quickly and produce hydraulic gradients across the system to such an extent that one section may fill completely and overflow before the whole system becomes full? If this were the case, then the question would be, where might control gates be placed in the storage treatment facility and how might they be operated so the facility could fill up uniformly?

2. Given the priority list for overflow locations defined in Chapter III, it was necessary to determine whether the system could be operated so the overflow sequence would occur as desired. One of the problems was that the sequence of overflows goes from west to east. Therefore, water must be moved to the extreme west end of the NSOC sewer during initial overflow periods. The question to be answered is if the Baker Street overflow gates were opened, what kind of gradients would be created throughout the system?

The approach to the hydraulic analysis was as follows. First, the hydraulic behavior of the system was studied under the simplest hydraulic system possible with the simplest operational control strategy. This approach meant that there would be no internal control gates; outfalls were considered to be weir outfalls, and the North Shore Pump Station was assumed to operate according to the simple rule described in Chapter IV. Furthermore, it was assumed that the Marina and the Embarcadero legs of the transport-storage system were hydraulically connected, i.e., there was no physical separation of the two legs by walls or gates. Thus, water could flow freely from one leg to another. These initial simulation studies would be used to evaluate the design of the system with regard to its hydraulic performance. Depending on the outcome of the simulation of this simple system, the operating rule for the pump station could be modified and control devices could be located as required to achieve the desired hydraulic response of the system to storm runoff.

The second part of the hydraulic analyses involved a study of hydraulic performance under overflow conditions. For these conditions a storm was selected that was expected to cause an overflow. The simple system was then simulated to see whether the system would first overflow at Baker Street and then sequentially overflow at Beach Street, Sansome Street and Jackson Street as intended. Where this was not achieved the model was used to determine the location of required internal control devices and operating rules for these control devices.

Prototype Representation

The design fo the NSOC transport-storage facility underwent several modifications in the course of this study. Figure V-2 shows an earlier version of the facility that was the starting point for the hydraulic analysis described here. Subsequent to the analysis, major design modifications were made, including the replacement of the rectangular section along North Point Street with a circular tunnel and the enlargement of the entire tunnel from 9 feet to 17 feet in diameter.*

The link-node schematic shown in Figure V-3 was used in the initial simulations. Camparing this figure with Figure V-2, we see that the 100 series nodes and links correspond to the Embarcadero leg of the NSOC sewer,

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*Subsequent design changes have reduced the tunnel diameter to 9 feet.

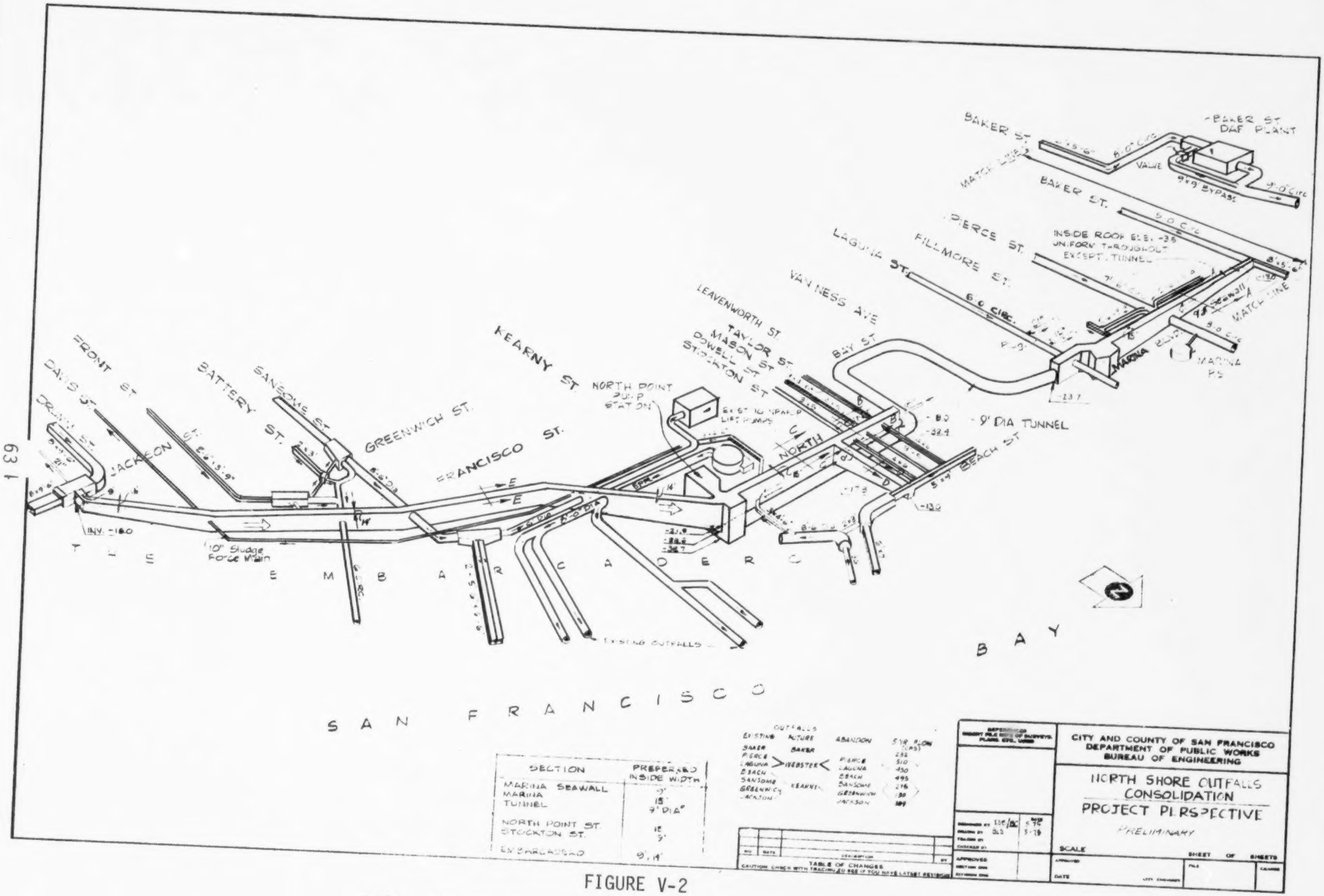

BAKER ST.
BAKER ST. DAF PLANT
VALVE
9'x9' BYPASS
9'-0" CIRC
MATCH LINE
BAKER ST.
5.0 CIRC
PIERCE ST.
INSIDE ROOF ELEV. -3.5
UNIFORM THROUGHOUT
EXCEPT TUNNEL
LAGUNA ST.
FILLMORE ST.
6.0 CIRC.
VAN NESS AVE
BAY ST
MARINA BLVD.
MARINA P.S.
MATCH LINE
-23.7
LEAVENWORTH ST.
TAYLOR ST.
MASON ST.
POWELL ST.
STOCKTON ST.
KEARNY ST.
NORTH POINT PUMP STATION
NORTH
-8.0
-32.4
9' DIA TUNNEL
BEACH ST
-13.0
-21.0
-38.6
-38.7
FRONT ST
DAVIS ST.
DRUMM ST.
JACKSON ST.
BATTERY ST.
SANSOME ST.
GREENWICH ST.
FRANCISCO ST.
6.0 CIRC.
INV. -16.0
10" Sludge Force Main
EXISTING OUTFALLS
THE EMBARCADERO
SAN FRANCISCO BAY
SECTION
PREFERRED INSIDE WIDTH
MARINA SEAWALL
9'
MARINA
15'
TUNNEL
9' DIA
NORTH POINT ST.
15'
STOCKTON ST.
9'
EMBARCADERO
9', 14'
OUTFALLS
EXISTING
FUTURE
ABANDON
5 YR FLOW (CFS)
BAKER
PIERCE
LAGUNA
BEACH
SANSOME
GREENWICH
JACKSON
BAKER
WEBSTER
KEARNY
PIERCE
LAGUNA
BEACH
SANSOME
GREENWICH
JACKSON
232
510
450
495
276
130
189
TABLE OF CHANGES
CAUTION: CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION
CITY AND COUNTY OF SAN FRANCISCO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING
NORTH SHORE OUTFALLS CONSOLIDATION
PROJECT PERSPECTIVE
PRELIMINARY
DESIGNED BY
DRAWN BY
TRACED BY
CHECKED BY
APPROVED
SCALE
SHEET OF SHEETS
DATE
FILE

FIGURE V-2

NSOC PROJECT PERSPECTIVE USED FOR INITIAL SIMULATION IN THE REAL TIME CONTROL STUDY

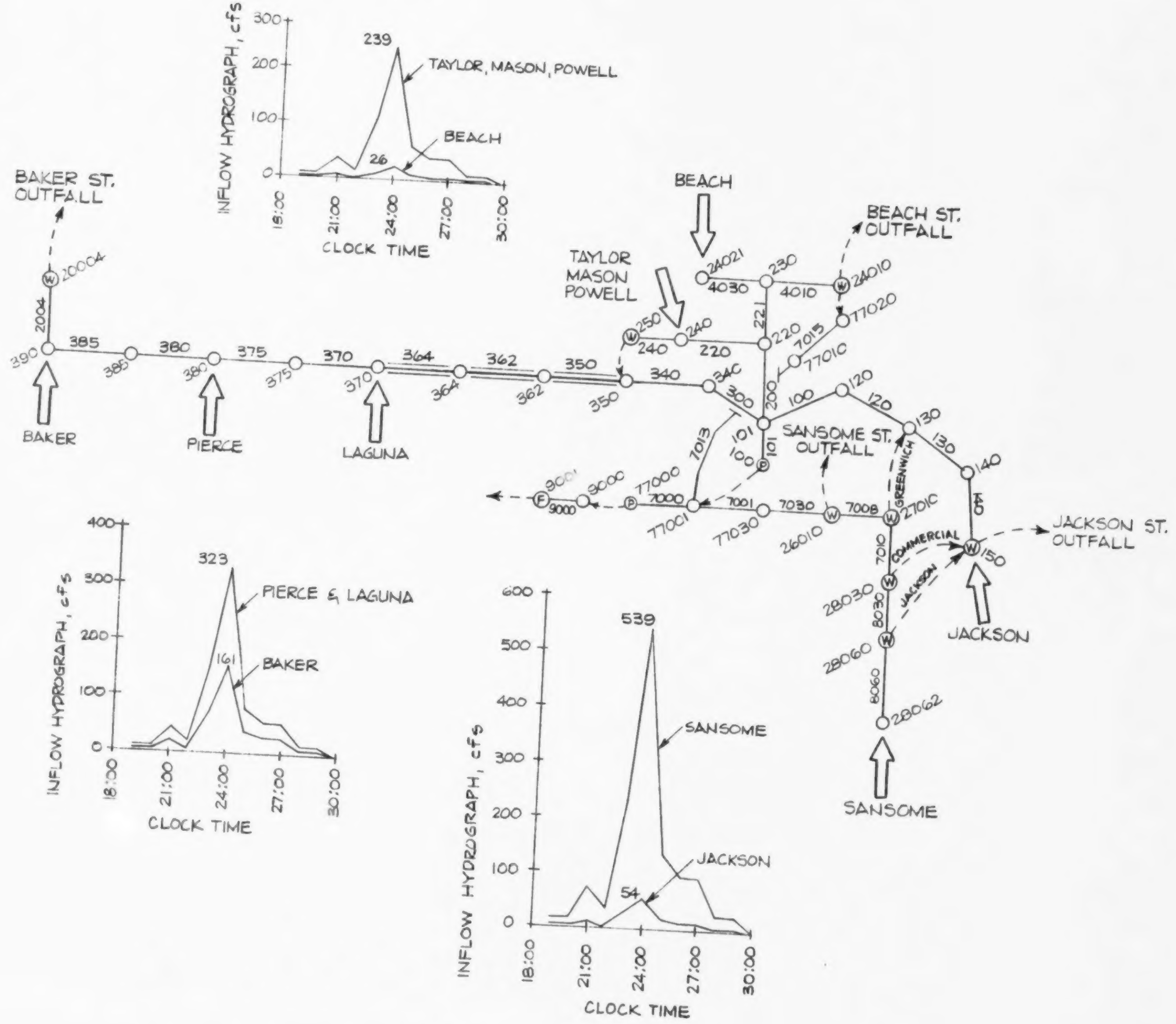

INFLOW HYDROGRAPH, cfs
300
200
100
0
239
TAYLOR, MASON, POWELL
BEACH
26
18:00
21:00
24:00
27:00
30:00
CLOCK TIME
BAKER ST. OUTFALL
20004
2004
390
385
385
380
380
375
375
370
370
364
364
362
362
350
350
BAKER
PIERCE
LAGUNA
TAYLOR MASON POWELL
250
240
240
220
340
34C
300
BEACH
24021
4030
230
4010
221
220
24010
BEACH ST. OUTFALL
77020
7015
77010
200
100
120
120
130
130
140
140
101
101
100
SANSOME ST. OUTFALL
7013
77000
7000
77001
7001
7030
77030
26010
7008
27010
GREENWICH
COMMERCIAL
7010
28030
8030
JACKSON
28060
8060
28062
150
JACKSON ST. OUTFALL
JACKSON
SANSOME
9001
9000
9000
INFLOW HYDROGRAPH, cfs
400
300
200
100
0
323
PIERCE & LAGUNA
161
BAKER
18:00
21:00
24:00
27:00
30:00
CLOCK TIME
INFLOW HYDROGRAPH, cfs
600
500
400
300
200
100
0
539
SANSOME
JACKSON
54
18:00
21:00
24:00
27:00
30:00
CLOCK TIME

FIGURE V-5

INITIAL SCHEMATIC REPRESENTATION OF NSOC SYSTEM AND INFLOW HYDROGRAPHS and the 300 series nodes and links correspond to the Marina leg, including the low level sewer portion from the tunnel outlet (node 350) to the North Shore pumping station (node 101). The 200 series is the upper level Beach Street leg. The 4000 series conduits are the existing Beach Street interceptor. The 7000 and 8000 series conduits are the existing Sansome Street line except for conduits 7013 and 7015 which connect the Beach Street interceptor to the North Point treatment plant.

The North Shore Pump Station (NSPS) is represented by Node 100. Intakes were located in the bottom of the NSOC sewer, and the discharge was to node 77001, which is about 100 feet upstream of the influent to the North Point Water Pollution Control Plant (NPWPCP). The pumps shown at node 77000 are the NPWPCP lift pumps which are located downstream of the grit chambers. Conduit 9000, which is shown in the schematic, is a dummy conduit, created to transport the discharge from the lift station out of the simulated system.

The storm of January 29-30, 1968, was selected for testing in the initial simulations because it was of moderate intensity and produced sufficient runoff to just fill the system storage without overflowing. The inflow hydrographs were developed by applying runoff coefficients to measured short-duration rainfall and are shown on Figure V-3.

Initial Simulations

Referring to Figure V-2, there are two interceptor lines that are not intercepted by the NSOC sewer; the Sansome Street line (pipes 7001, 7030, etc.) and an egg shape sewer (pipes 7013 and 7015) connecting Beach Street to the North Point treatment plant. Initially it was planned to keep both of these direct connections until the NPWPCP was abandoned so that dry weather flow from the Sansome Street and Beach Street lines would not have to be relifted through the NSPS.

Figure V-4 shows the simulated flows in these interceptors as a result of routing the test storm through the North Shore system. Notice that a negative flow occurred in the Beach Street connection. This means that during wet weather operation, this pipe (conduit 7013) would divert flows away from the treatment plant toward Beach Street. Figure V-4 shows that the reverse flow in conduit 7013 reached nearly 40 cfs. The result is a backup in the Beach Street line and a potential overflow at the Beach Street outfall. As a result of this finding, it was decided to terminate the Beach Street connection at its intersection with the NSOC sewer and to block off the line between junction 77001 and the NSOC sewer.

Following the decision to block conduit 7013, the system schematic was revised as shown in Figure V-5.

Analysis of North Shore Pump Station Operating Rules

The initial simulation indicated that the North Shore Pump Station would shut down when the flow in the Sansome Street sewer (conduit 7001) reached 232 cfs (See Figure V-4). In other words, the gravity flow in the Sansome Street sewer would be at the capacity of NPWPCP and pumping from the NSOC facilities would be stopped. This means that there would be essentially no outflow from the lower part of the NSOC transport-storage system and it would function purely as a storage basin. It was decided to examine as an alternative an operating rule for the NSPS which allowed pumping from the NSOC system simply on the basis of head at node 100. An explicit statement of the two operating rules for the NSPS is:

RULE 1 - Give priority for treatment to wet weather flow in Sansome Street Line

$$Q_{NSPS} = f[(232 - Q_{7001}), H_{100}]$$

$$= \text{Minimum of } (232 - Q_{7001}) \text{ and } Q_{ALLOW}$$

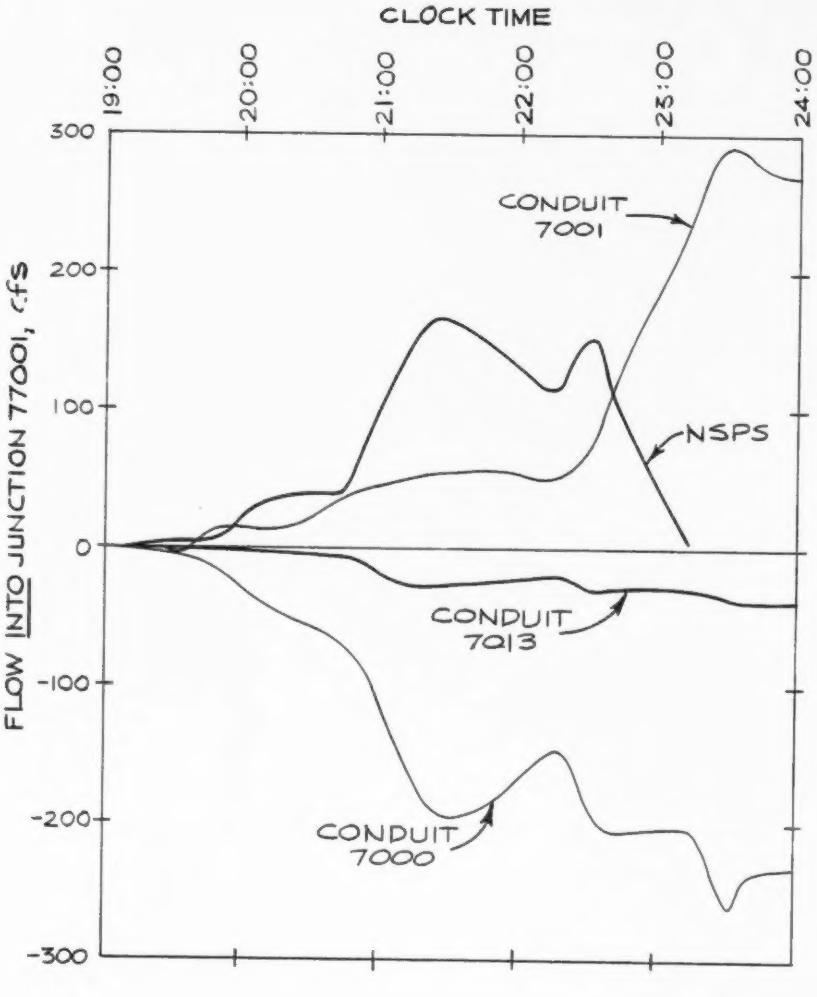
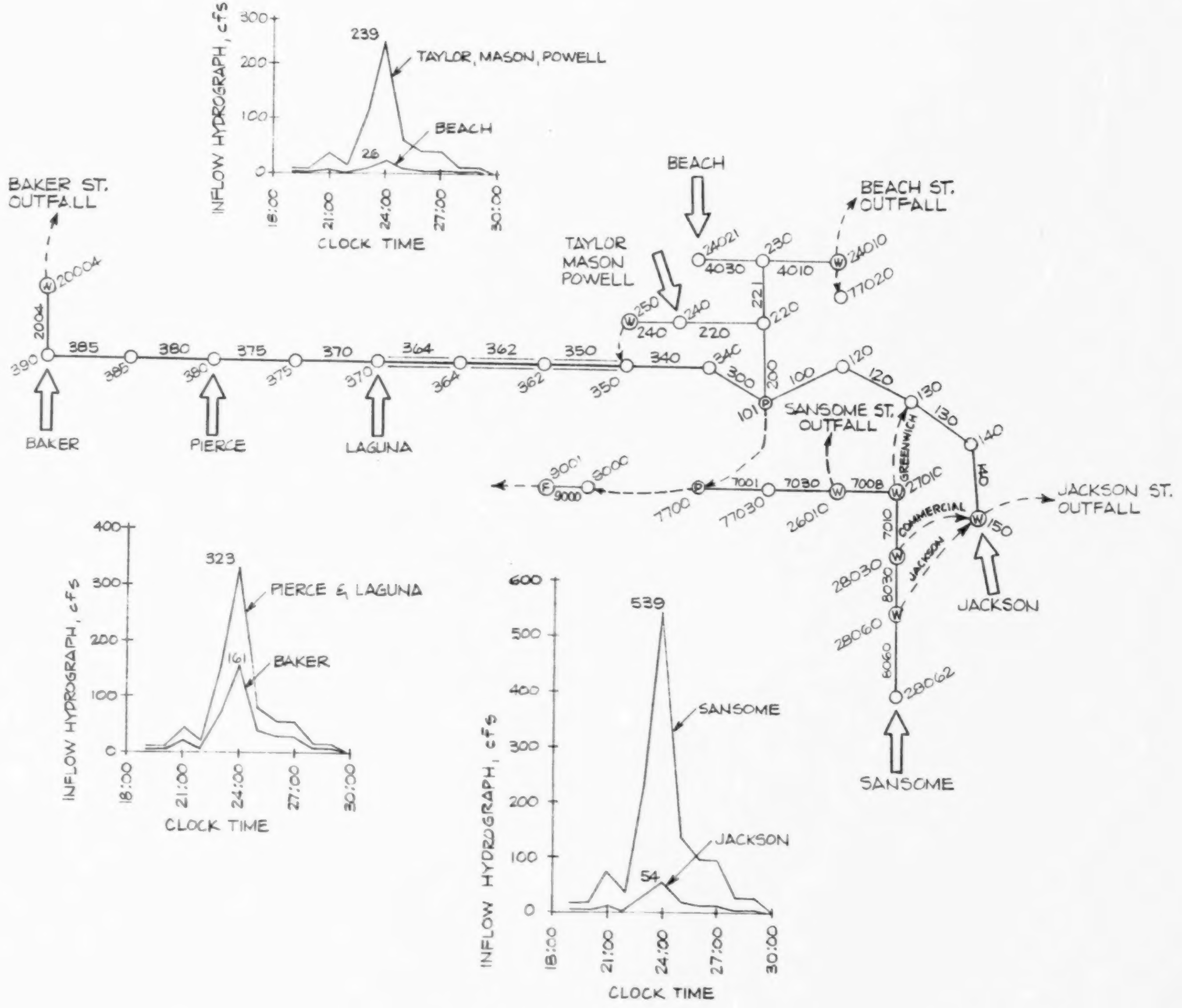


FIGURE V-4

FLOW BALANCE AROUND JUNCTION 77001, FROM INITIAL NSOC SIMULATIONS


INFLOW HYDROGRAPH, cfs
300
200
100
0
18:00
21:00
24:00
27:00
30:00
CLOCK TIME
239
TAYLOR, MASON, POWELL
BEACH
26
BAKER ST. OUTFALL
20004
2004
390
385
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375
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362
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340
34C
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200
101
BAKER
PIERCE
LAGUNA
TAYLOR MASON POWELL
250
240
220
221
BEACH
24021
4030
230
4010
24010
77020
BEACH ST. OUTFALL
100
120
130
140
150
SANSOME ST. OUTFALL
GREENWICH
27010
COMMERCIAL
JACKSON
JACKSON ST. OUTFALL
3001
9000
8000
7700
7001
77030
7030
26010
7008
7010
28030
8030
28060
8060
28062
SANSOME
INFLOW HYDROGRAPH, cfs
400
300
200
100
0
18:00
21:00
24:00
27:00
30:00
CLOCK TIME
323
PIERCE & LAGUNA
161
BAKER
INFLOW HYDROGRAPH, cfs
600
500
400
300
200
100
0
18:00
21:00
24:00
27:00
30:00
CLOCK TIME
539
SANSOME
JACKSON
54

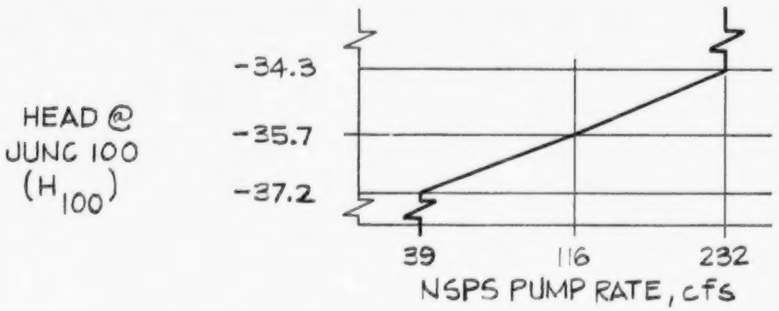
FIGURE V-3

REVISED NSOC SCHEMATIC AND INFLOW HYDROGRAPHS where

Q_{NSPS} = pumping rate from Node 100 to Node 77000,

Q_{7001} = flow in the Sansome Street sewer (conduit 7001), and

Q_{ALLOW} = maximum possible rate at which the NSPS can pump as defined by the following rule curve



RULE 2 - Give priority to treatment of wet weather flow in the NSOC sewer

$$\begin{aligned} Q_{NSPS} &= f(H_{100}) \\ &= Q_{ALLOW} \end{aligned}$$

Note that in both cases--Rule 1 and Rule 2--the flow in the Sansome Street sewer is solely gravity controlled as determined by the inflow hydrograph at Junction 28062 and the head at Junction 77000.

A comparison of flows around Junction 77000 for the two cases is shown in Figure V-6. Notice that the treatment rate (NPWPCP on the right hand graph) which is a function of the head at Junction 77000 only, is the same under both NSPS operating rules. The NSPS response under the two operating rules is shown on the right hand figure while the flow in the Sansome Street Sewer is shown on the left. Note that the system responds identically under both rules until the sum of Q_{7001} plus Q_{NSPS} is greater

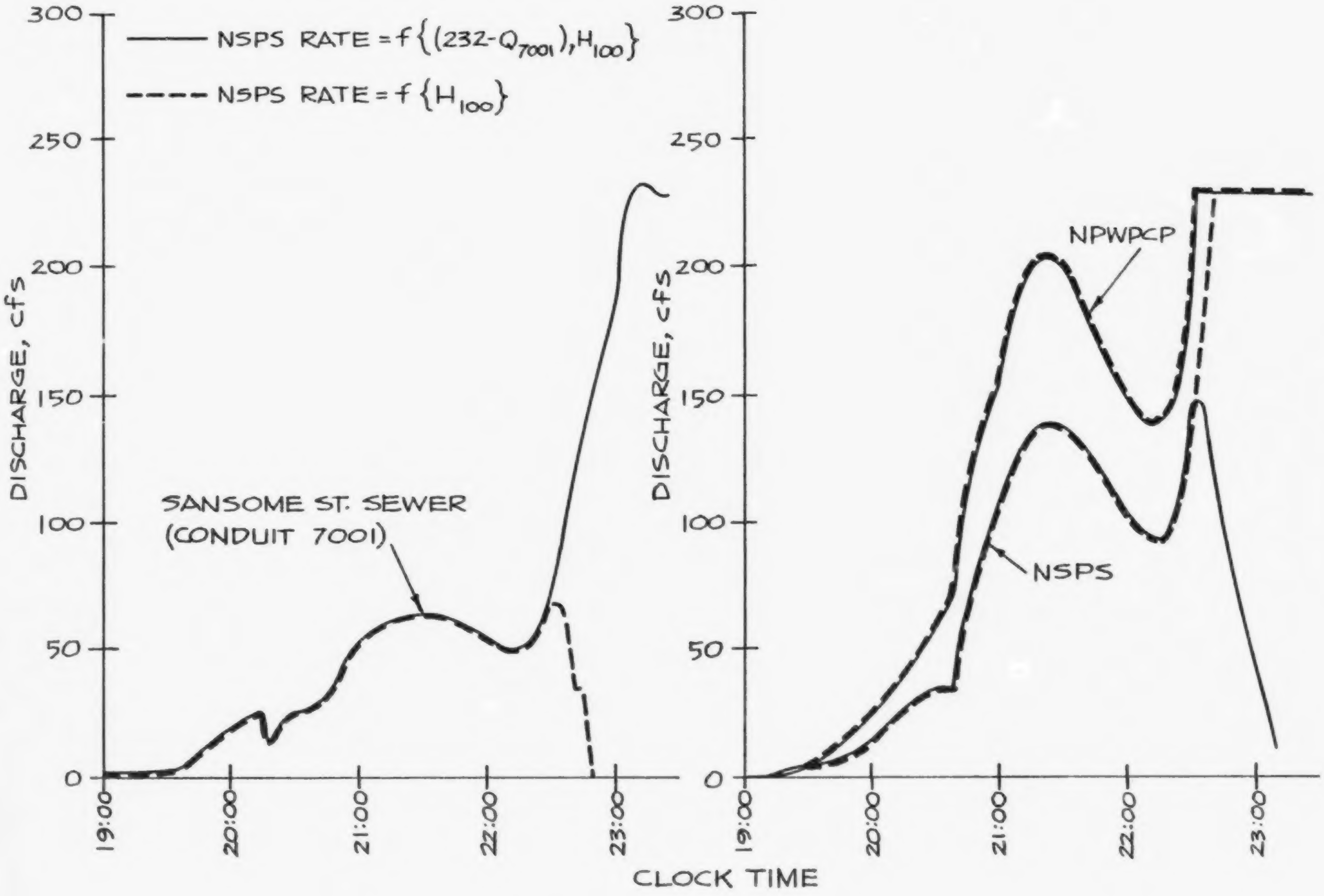


FIGURE V-6

COMPARISON OF NSPS AND SANSOME STREET SEWER RESPONSE UNDER
TWO DIFFERENT NSPS OPERATING RULES, FROM INITIAL NSOC SIMULATIONS

than 232 cfs which occurs at approximately 22:30. After this time, the NSPS shuts down under Rule 1 and the Sansome Street line continues to flow; under Rule 2 the exact opposite happens.

It was preliminarily decided not to allow the NSOC sewer to stop flowing. This consideration is based primarily on hydraulic factors that must be taken into account in turning on the pump station, shutting it down, and then later turning it on again. Thus, the original operation rule (Rule 1) was modified as a result of this analysis and Rule 2 was preliminarily adopted.

Under Rule 2, the Sansome Street interceptor backs up as the North Shore Pump Station comes on-line. The left hand graphs on Figure V-7 shows the inflow hydrograph to the Sansome Street line and the outflow through conduit 7001. The difference in these two hydrographs will first go into conduit storage and then will overflow at one or more of the weir locations shown on Figure V-5. The distribution of the overflows is shown on the right hand graph on Figure V-7. By far the largest overflow is at the Sansome Street Outfall, which is unacceptable since it constitutes a system overflow. The other three overflows are intercepted by the NSOC sewer and therefore are not a problem. As a result, some kind of device will have to be installed at the Sansome Street outfall to prevent these uncontrolled overflows from occurring.

Hydraulic Profiles in the NSOC Sewer

Figure V-8 shows the hydraulic profile in the NSOC sewer at three different times. At 22:45, the NSPS has just reached its capacity; the water surface profile shows that backwater in the system is very slight. At this point in time, system inflows are 600 cfs and rising while outflows from the NSPS will remain constant from this time forward at 232 cfs. The result is a buildup of backwater in the system. By 23:15 the head had risen to

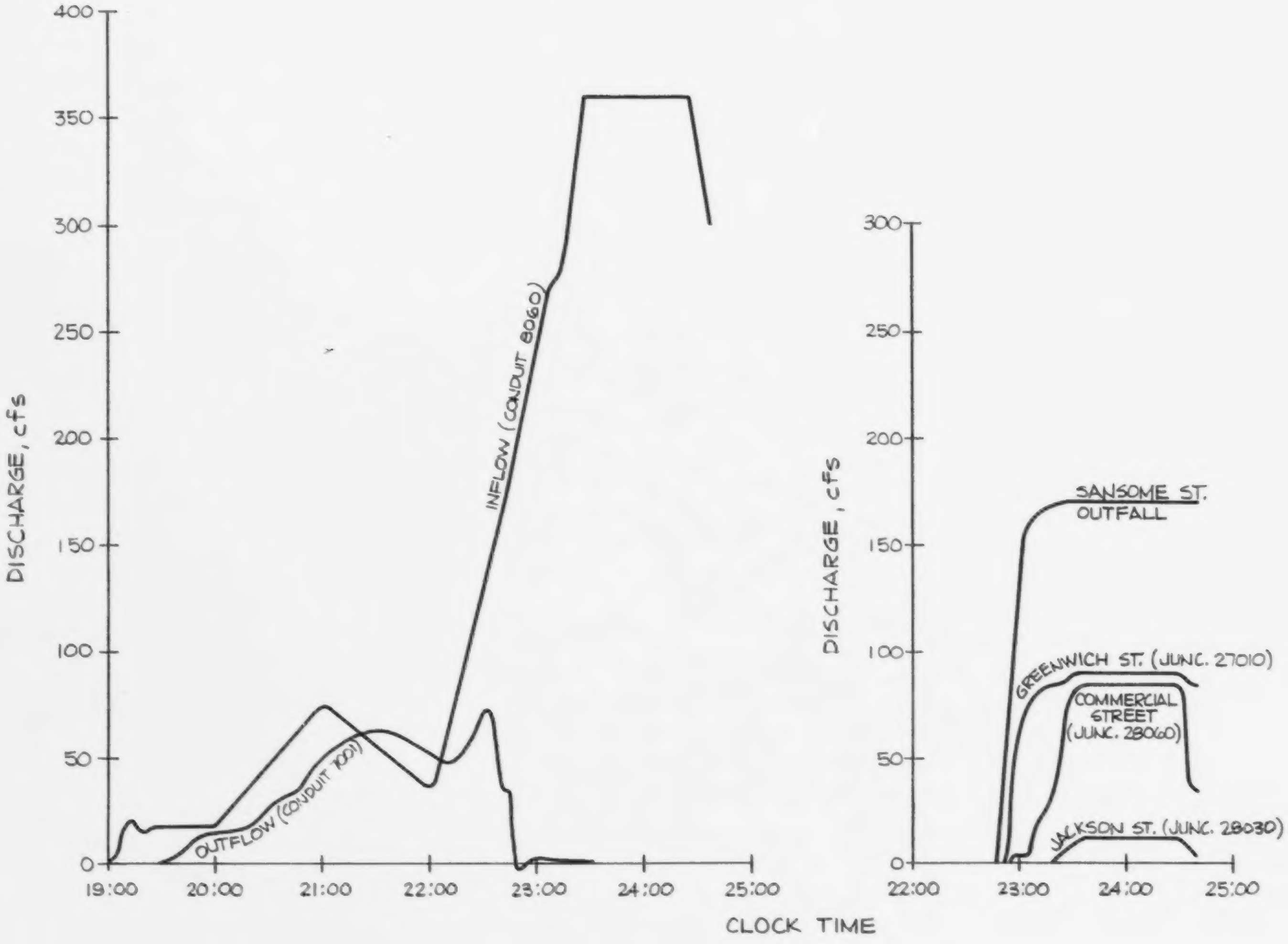


FIGURE V-7

HYDRAULIC RESPONSE OF SANSOME STREET SEWER TO SYSTEM OPERATION UNDER NSPS RULE 2, FROM INITIAL NSOC SIMULATIONS

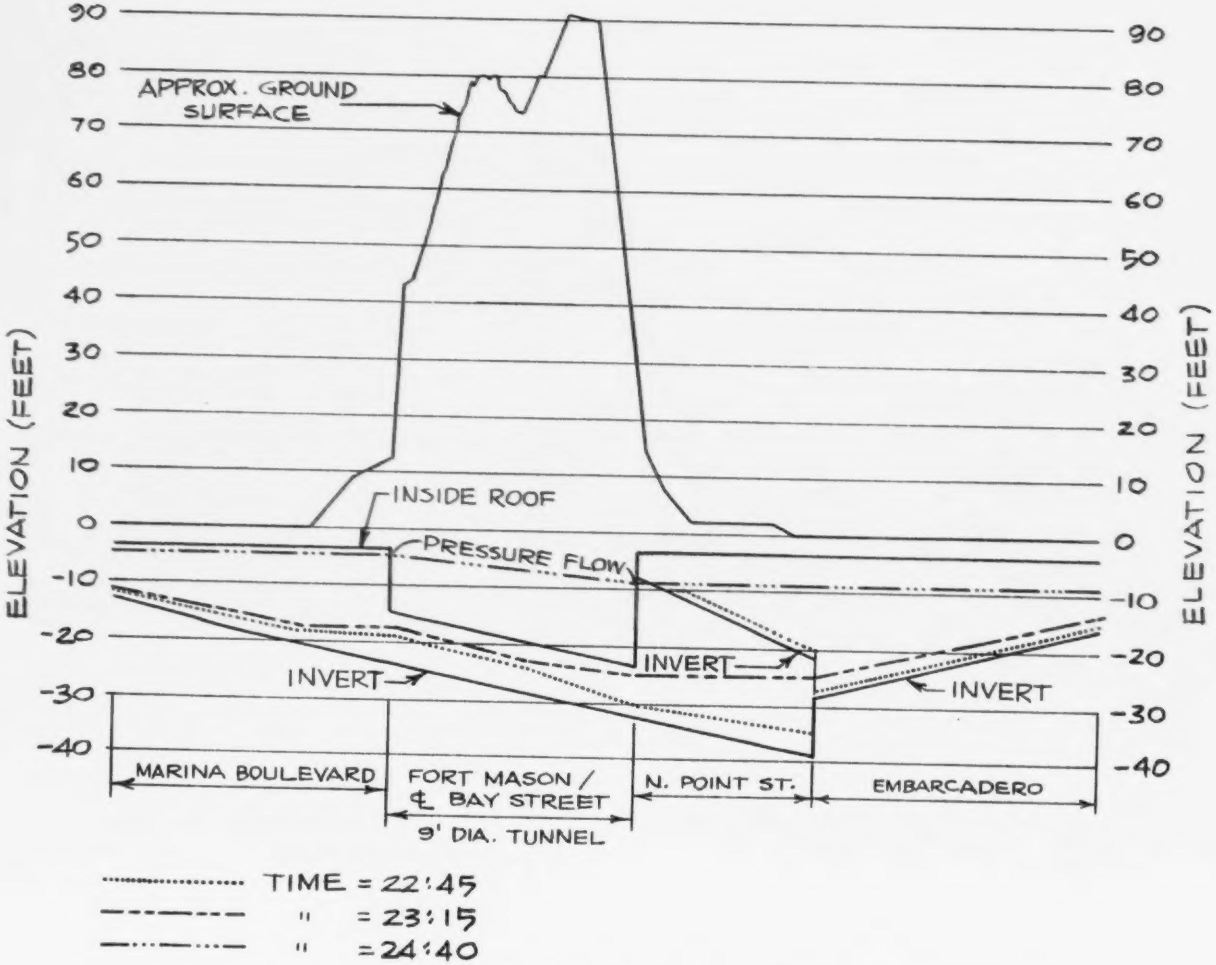


FIGURE V-8

HYDRAULIC PROFILES IN NSOC SEWER FOR SYSTEM OPERATION UNDER NSPS RULE 2, FROM INITIAL NSOC SIMULATIONS elevation -25 feet and the backwater extends from the NSPS almost halfway through the tunnel on the Marina Street section and about halfway up the Embarcadero section of the NSOC sewer. By 24:40 the entire system is backwater affected.

It is interesting to note that the hydraulic profile is nearly flat as the water surface approaches the top of the NSOC sewer; however, there is a significant headloss through the tunnel section of the Marina branch. The main flow control in the Marina leg of the sewer is the tunnel. Thus for practical purposes, we can assume that once the tunnel is full, the NSOC sewer can be conceptualized as three storage chambers. Two are linked by the tunnel on the Marina leg and the third (the Embarcadero leg) is operationally linked to the other two by the NSPS which requires that the sum of the outflows from the Embarcadero leg and lower Marina leg not exceed 232 cfs.

Wave Propagation in the NSOC Sewer

In the process of simulating the hydraulic response of the NSOC Sewer a wave propagation problem was encountered in the TRANSPORT model. The problem resulted primarily from the high rate of flow propagating down the Marina leg through the tunnel and then finally decelerating and to a near zero flow at the lower end of the sewer. As a result of the deceleration in this section, a surface wave developed which would propagate back up through the sewer amplifying as it progressed toward Baker Street. This wave would further amplify as it was reflected back downstream toward the pump station. The wave amplification would eventually cause the TRANSPORT model solution to become unstable. The question of wave propagation was studied in some detail and a decision was made that the problem should be studied in a physical model.

A physical model of the system was built by Hydro Research Science, Santa Clara, California. The physical model studies confirmed that the wave problem did indeed exist, and modifications to the prototype were recommended by Hydro Research Science to dampen the effect.

Because of the wave propagation problem it was never possible to study the performance of the facility under overflow conditions. The simulation, however, was not really necessary at this point in the study although it could have been accomplished by starting the simulation when the storage-transport facility was nearly full. Because of the flatness of the water surface in the open-channel portions of the storage transport facility it was decided that to operate the outfalls on a priority basis desired would not be a problem. The question revolved more about what kind of control devices should be used rather than how they should be operated.

Summary

In summary, the results of the hydraulic analysis of the NSOC project showed that some of the proposed features in the design would not work as projected, mainly the connection from Beach Street to the North Point Treatment Plant. Also, giving priority at the treatment plant to runoff from the Sansome Street interceptor rather than to the North Shore Pump Station caused the NSPS to be shut down and a stagantion condition to occur in the NSOC sewer any time the interceptor flow was more than 232 cfs. These findings resulted in significant savings to the City in terms of the cost of corrective measures that would have had to be taken had the results not been discovered early in the design phase. The hydraulic analysis was also useful in identifying the wave problem in the Marina leg of the sewer which would have been extremely expensive to correct once the prototype had been built. Finally, in terms of the RTAC system, the hydraulic analysis showed that a sophisticated simulation model was not needed for the NSOC sewer and that a simple storage routing model would be quite adequate to simulate the behavior of the storage transport facility.

The results of applying the TRANSPORT model in this study were not as glamorous or as exciting as had been hoped for, but a number of useful points were learned that are generally applicable, we believe, to the study of any storm drainage system or storm water management system which is intended to have some type of flow control devices in it.

First of all, it is necessary to use a model like the TRANSPORT model to study the hydraulic response of the system to typical storms which can be expected on the area. This study enables one to determine to what extent the system can be controlled, where control devices and storage basins should be located in the system for best hydraulic performance and how the flow control devices and storage devices should be operated to minimize uncontrolled overflows from the system. *In short, the TRANSPORT model is an essential tool for determining the operating rules of a RTAC system.*

With regard to actual use of the TRANSPORT model to simulate the hydraulic response of the system in real time, two points are relevant. First, the model produces a more sophisticated solution than is required for such purposes since the results generated are more precise than the accuracy of the data than can be put in. Secondly, and maybe more importantly, the basic solution technique used in the TRANSPORT model can become unstable under certain, not always known, sets of hydraulic conditions. The situation is intolerable in a real time automatic control system where it is necessary to have an estimate of the hydraulic response rather than an inoperative mathmatical model. This reason alone is sufficient to preclude its use as the RTAC routing simulator for the NSOC System. Once the operating rules for the system are known, it would be better to develop a simplified storage routing model that would represent the anticipated or projected response of the system (flows and heads) to currently occurring rainfall or predicted rainfall.

VI. RAINFALL PREDICTION AND REAL TIME CONTROL

INTRODUCTION

The capability to forecast expected rainfall during a storm may be significant importance in the operational optimization of transport, storage and treatment facilities for real-time control of combined sewer flows. In conjunction with the real-time monitoring of system-hydraulic status and the operational constraints imposed on the system, such a forecast capability could lead to the timely manipulation of control devices and the decision on actual start-up of treatment plants and pumping stations.

Rainfall prediction may also be applied, through the use of a model, to historic rainfall data for the predictive reconstruction of past storms. The latter can be used in the evaluation of the rainfall prediction model itself and the development and testing of control strategy alternatives.

This chapter describes the development of the rainfall prediction model, RAFORT, as it relates to the storm characteristics and rainfall data availability in the San Francisco area. It discusses the reasons for selecting the adopted approach to rainfall prediction and describes the prevailing storm characteristics and rainfall data requirements, availability and analysis. The concept and theory of the model RAFORT are briefly described. The results from a sample prediction are presented and explained and the applications of two offspring forms of RAFORT are discussed. The chapter concludes with a discussion on rainfall prediction accuracy in relation to the use of such a prediction and the constraints imposed on the physical system to be operated. More detailed information about RAFORT is contained in the report "Rainfall Prediction" prepared for the City and County of San Francisco, San Francisco Wastewater Program, by Christos A. Phanartzis, March 1979.

RAINFALL CHARACTERISTICS

Regional Aspects

Rainfall in the San Francisco area is dominantly winter precipitation and the months of November to April account, on the average, for 90 percent of the annual rainfall, January usually is the wettest month. In excess of 90 percent of the precipitation is attributed to cyclonic storm systems generated in the Northern Pacific Ocean and entering Northern California from the West and Northwest.

As reported by other investigators and confirmed by local rainfall data, most of these cyclonic storms have durations from a few hours to four or more days and are characterized by a banded structure. The bands making up the storm system move rapidly and are generally parallel to the storm front converging mildly toward the low pressure region of the system. The bands account for the high intensity ranfall periods which are superimposed on a more uniform low intensity or zero rainfall background.

The bands are themselves made up of a number of individual short-lived cells, or groups of overlapping cells, of high rainfall activity. The cells have diameters on the order of two to five miles and displace themselves in a general direction parallel to the movement of the bands, their development, dissipation and regeneration being random. These cells are responsible for the localized, high intensity rainfall over short periods (5-10 minutes) at different times during a storm.

Local Aspects

San Francisco has a continuous record of rainfall observations since 1907 at the National Weather Service (NWS) station at the Federal Office Building (FOB). The same agency, now under the National Oceanic and Atmospheric Administration (NOAA), has been operating a raingage on the

West Side of the City since 1948. In 1971, the City and County of San Francisco established its own network of 30 automatic, continuously reporting raingages corresponding approximately to one raingage per 1.5 sq. miles (3.9 sq. km.). Two remote raingages in neighboring Marin County were installed in 1976 and 1977 respectively by the City of San Francisco as part of its data acquisition network. Figure VI-1 shows the location of these remote raingages as well as five other stations that could be used in subsequent rainfall prediction efforts. Simultaneous rainfall reports from both the remote and local gages have contributed greatly to the refinement of the real-time rainfall prediction model described in this chapter.

The 70-year record for the FOB raingage has provided valuable information in understanding and determining the long-term characteristics of point rainfall. The 70-year mean annual rainfall at this gage is 20.22 inches (514 mm).

The data from the 30-gage in-city network has operated since 1971 and enabled both spatial and temporal characteristics of City-wide rainfall to be studied. Based on this relatively short record, the City of San Francisco was found to receive an average of 1.12 times the rainfall at the FOB raingage. The long-term average spatial pattern and variability of rainfall in the city is illustrated in the isohyetal map of Figure IV-2. For short periods of time during the same storm event, the spatial variation is even more pronounced. The spatial variation of rainfall for different durations is shown in Figure VI-3. The figure shows that short duration storms experience higher spatial variations than long duration storms. The dashed curve in this figure is based on the last six years of rainfall data and represents the annual average picture as described by the isohyets in Figure VI-2. In the real time control of combined sewer flows the spatial variation of short-duration rainfall becomes of particular importance.

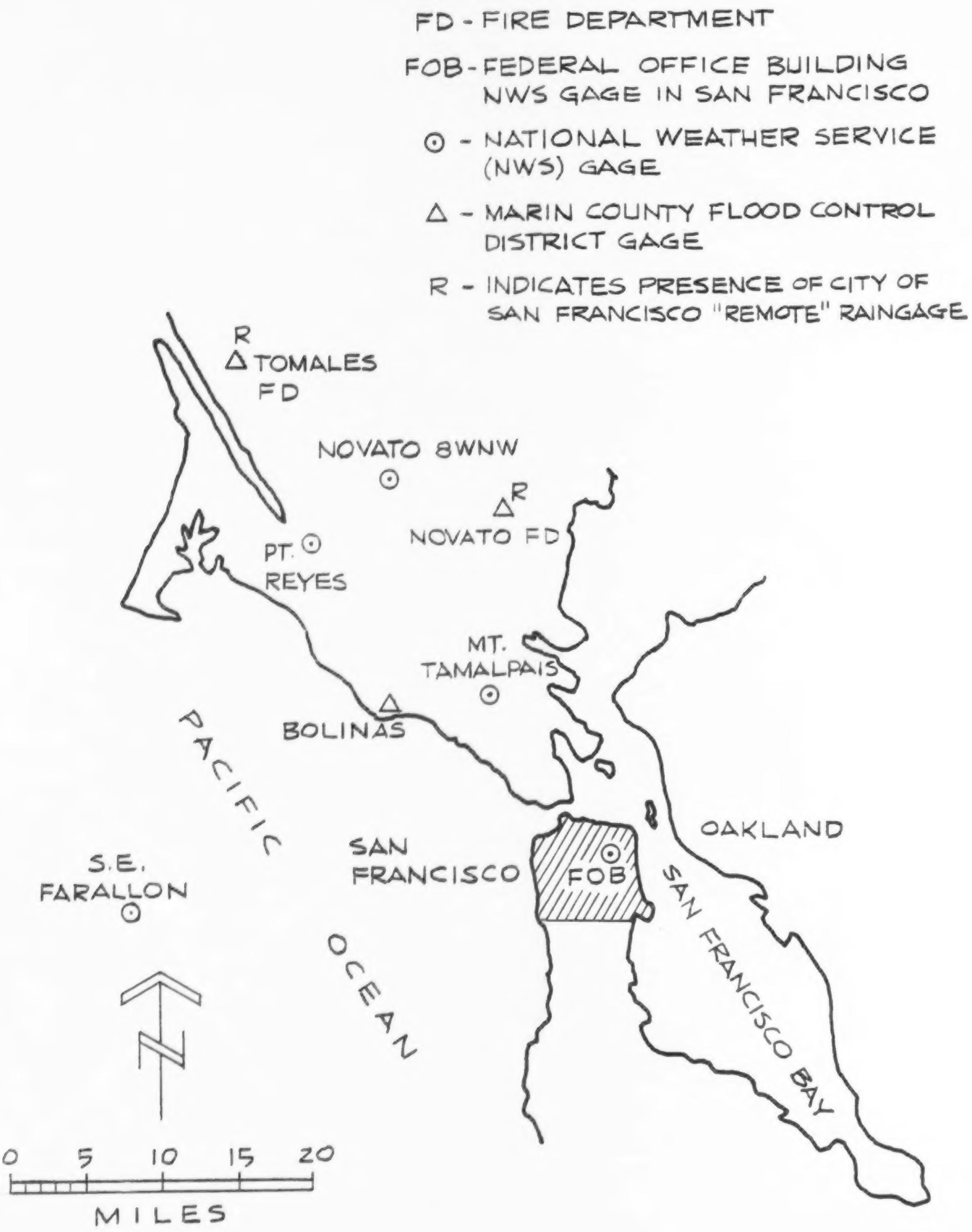

FD - FIRE DEPARTMENT
FOB - FEDERAL OFFICE BUILDING NWS GAGE IN SAN FRANCISCO
⊙ - NATIONAL WEATHER SERVICE (NWS) GAGE
△ - MARIN COUNTY FLOOD CONTROL DISTRICT GAGE
R - INDICATES PRESENCE OF CITY OF SAN FRANCISCO "REMOTE" RAINGAGE
R
TOMALES FD
NOVATO 8WNW
R
NOVATO FD
PT. REYES
MT. TAMALPAIS
BOLINAS
PACIFIC OCEAN
SAN FRANCISCO
FOB
OAKLAND
SAN FRANCISCO BAY
S.E. FARALLON
0
5
10
15
20
MILES

FIGURE VI-1

LOCATION OF RAINGAGES IN MARIN COUNTY, NORTH OF SAN FRANCISCO, AND SE FARALLON ON THE WEST

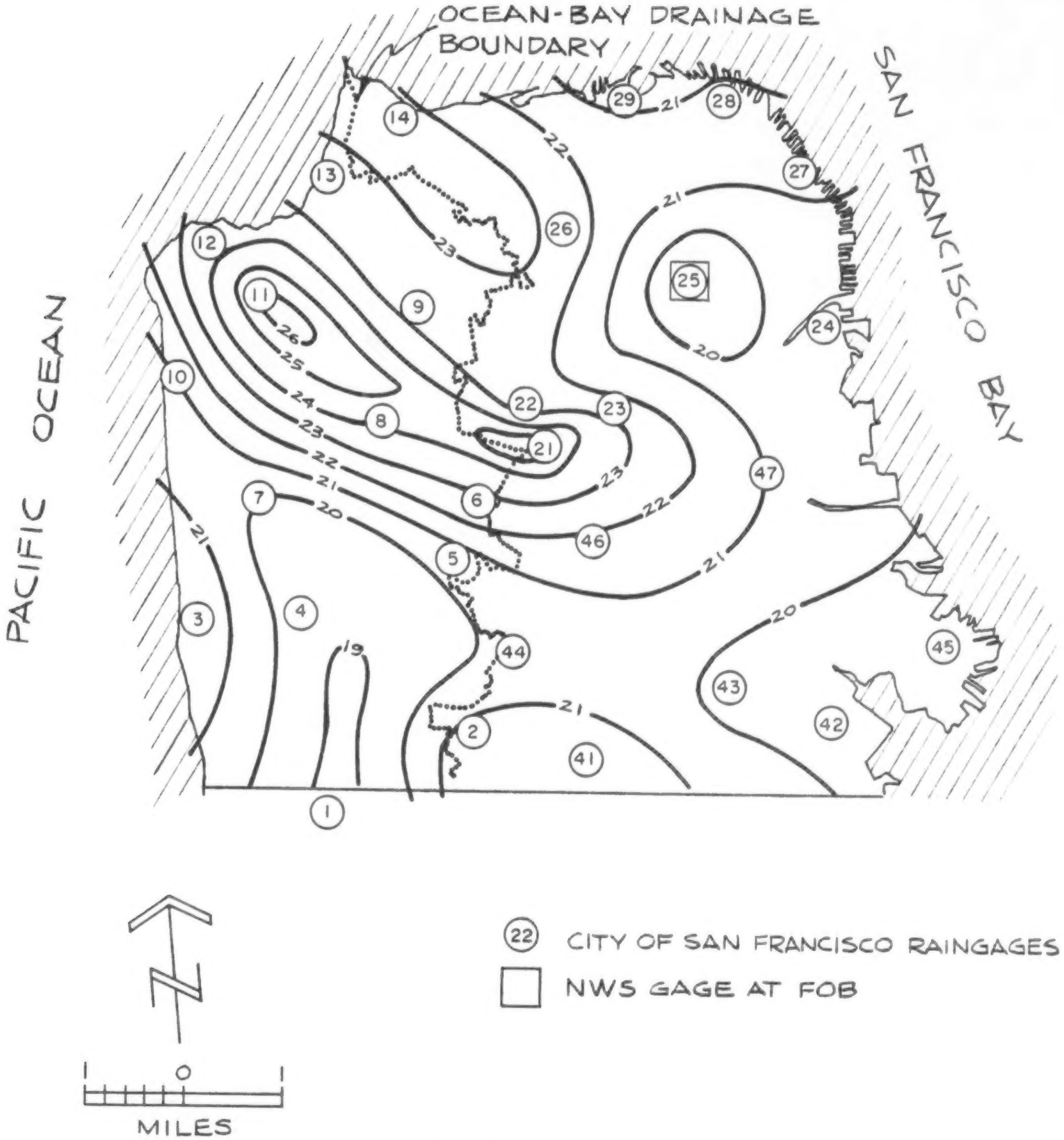

OCEAN-BAY DRAINAGE BOUNDARY
SAN FRANCISCO BAY
PACIFIC OCEAN
1
2
3
4
5
6
7
8
9
10
11
12
13
14
21
22
23
24
25
26
27
28
29
41
42
43
44
45
46
47
19
20
21
22
23
24
25
26
22 CITY OF SAN FRANCISCO RAINGAGES
NWS GAGE AT FOB
N
1
0
1
MILES

FIGURE VI-2

ISOHYETAL MAP OF SAN FRANCISCO SHOWING
AVERAGE ANNUAL RAINFALL IN INCHES, BASED ON RECORDS FOR 1972-1978
(1 inch = 25.4 mm.)

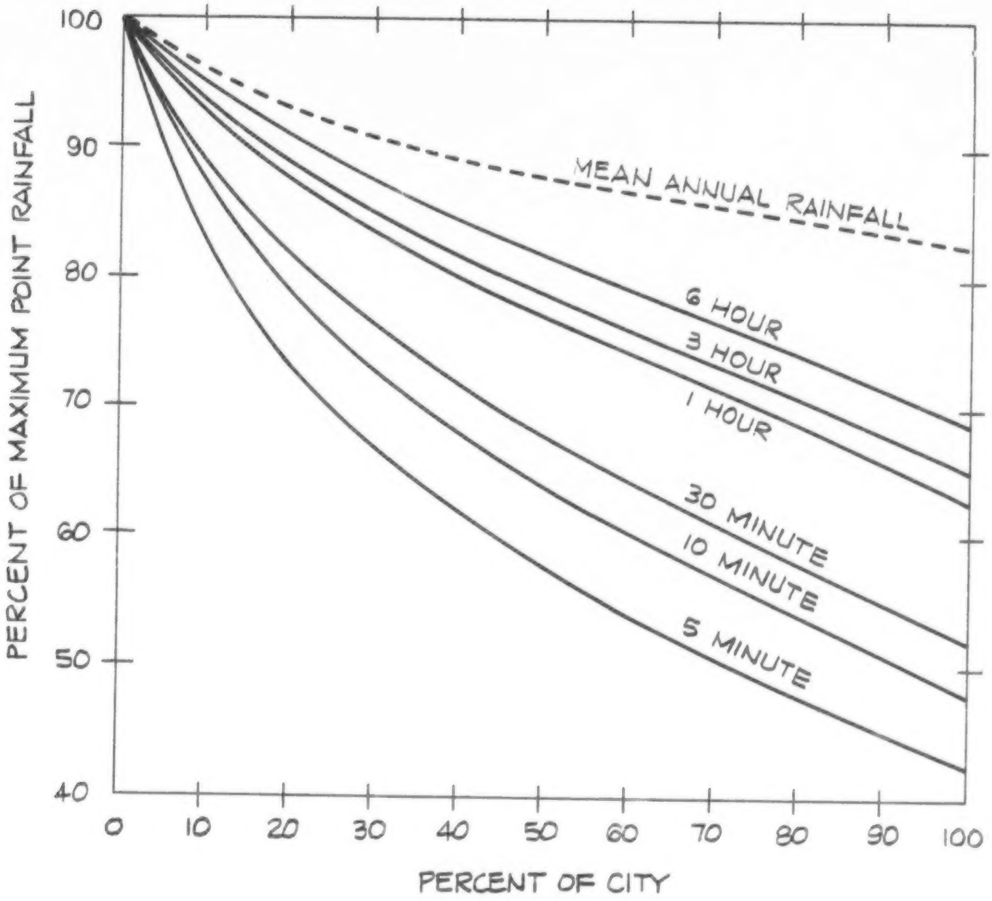


FIGURE VI-3

AREA-DEPTH CURVES FOR DIFFERENT RAINFALL DURATIONS, WITHIN THE SAME STORM BASED ON 1972-1978 STORM-MAXIMUM DATA IN SAN FRANCISCO

ALTERNATIVES FOR RAINFALL PREDICTION

It is obvious that the kind of rainfall predictions sought here should be underlain by the smallest possible uncertainty since such predictions are intended to form the basis for real time operational decision making.

The forecasts issued by the National Oceanic and Atmospheric Administration (NOAA) are based on satellite, radar and synoptic weather data and cover large areas and long periods of time (6 hours). For the purpose to be served in San Francisco these forecasts were found to be too gross and too generalized in time and space to be applicable over a small area.

The Illinois State Water Survey has embarked on a real time rainfall determination at different parts of the City of Chicago and its environs by using a combination of radar echoes and rainfall observations from a large network of 320 raingages. The role of the radar is not strictly of a predictive nature but rather to enhance the quality of real time areal rainfall estimates. However, the importance of radar in storm-tracking and pattern anticipation as they relate to short-term rainfall has been recognized both by Chicago and San Francisco.

Parallel to the effort by the City and County of San Francisco, Colorado State University (CSU) has examined the possibilities of real time rainfall prediction as it applied to urban stormwater management. The outcome of the CSU effort was an "autoregressive transfer function model" called FORCST. This model makes use of historic storms to determine the parameters of a base-line model. In a real time situation, as a storm progresses, FORCST used current rainfall data which are continuously becoming available to adjust model parameters. Base-line and currently derived model parameters can be weighted accordingly in the updating process. FORCST has the capability to produce forecasts of rainfall at a number of given sites or proceed to forecast expected inflows for these sites directly, i.e. going directly from rainfall to inflow instead of rainfall to rainfall and then to inflow through a separate rainfall-runoff model. FORCST is a "learning" model and the forecast is claimed to improve as the storm progresses.

Another class of models deals with the stochastic generation of synthetic storm sequences having statistical properties similar to the historic storm record. Although powerful tools for other hydrologic applications, those models were not pursued due to the uncertainity involved in the generation of a single storm event as it evolves in real time.

The rainfall prediction model RAFORT developed by the City and County of San Francisco may be classified as a stochastic-empirical model. Its similarity to the CSU FORCST model is in the use of both base-line model parameters and real time parameters derived from current rainfall data. RAFORT, uses real time remote rainfall data to forecast local rainfall over a future period of two to five hours, whereas FORCST uses similar data from a group of local gages and attempts to forecast rainfall for each interval (30-minutes) within the desired prediction period.

RAINFALL DATA AVAILABILITY

The type of rainfall prediction model chosen for this investigation called for rainfall data from local raingages as well as from distant locations in the direction of storm approach, mainly from the west and northwest of San Francisco.

Figure VI-1 shows the remote rainfall stations for which some form of pertinent rainfall data are available. The ideal situation is to have several dependable raingages at different distances from the area of interest and along the dominant direction of the storm approach. The Pacific Ocean west of the city unfortunately sets a constraint in this respect.

The rainfall data for the SE Farallon raingage some 30 miles West of San Francisco was too fragmental to be of any use in this study and was discarded.

Rainfall data is available in a variety of forms and formats from different agencies ranging from a national to a local level.

The National Climatic Center in Asheville, NC has provided hourly rainfall for a number of National Weather Service (NWS) gages in the form of magnetic tapes and monthly returns. The Department of Water Resources for the State of California compiled and published short-term (5 min. to 24 hr.) rainfall information for several NWS gages, including the FOB gage for which depth-frequency-duration studies were completed using data since 1907. For the raingages operated by Marin County Flood Control District (MCFCD) the data was available in the form of raingage charts, a selected number of which were analyzed manually on a 30-minute and one-hour basis.

The period of rainfall records for which most of the raingages had concurrent data at the time of analysis was confined to three winter seasons during the years 1972-1975. All raingages in Figure VI-1, except the NWS gage at FOB in San Francisco, suffer from occasional, and in some cases frequent data discontinuities during a number of storms. The most reliable remote raingages were Novato 8WNW and Tomales FD. Using the 1972-75 record, a file of 27 storms ranging from one to four days was assembled, containing rainfall data for the two remote gages, the FOB in San Francisco and areal rainfall for the North Shore area of San Francisco. The latter was derived by taking the weighted mean of six raingages in the area involved.

The 27-storm rainfall data set was subjected to correlation, regression and coincidence analysis for determining first, the feasibility of this approach to real time rainfall prediction and second, to derive base-line values of model parameters described and defined later in this report. It was concluded that hourly rainfall demonstrated higher correlation between local and remote rainfall than the 30-minute data. Lower correlations obtained with the 30-minute rainfall data reflects the high variability of the individual rainfall cells described earlier in the chapter. As a consequence, only hourly rainfall data has been used throughout the rainfall prediction effort.

THE RAINFALL PREDICTION MODEL: RAFORT

RAFORT, which stands for "Rainfall Forecasting in Real Time", is a real time version of a rainfall prediction model. Other specialized versions of this model, using the same prediction algorithms, were formulated at times to perform related functions and are described later in the chapter. RAFORT uses a deterministic process to arrive at a forecast and may be described as a stochastic-empirical model. It is stochastic in the sense that the way the parameters are estimated implies some degree of uncertainty. RAFORT does not provide for area allocation of the derived rainfall predictions.

Concept and Theory

The storm is assumed to be moving from an independent (remote) raingage X to the dependent (local) raingage Y. By cross-correlating simultaneous rainfall observations from the distant raingage X, with a local raingage (or mean of a group of gages) Y, it is possible to derive a short-term forecast of local rainfall at gage(s) Y.

The relation used in RAFORT takes the form:

$$Y_{t+k} = a + bx_t \tag{1}$$

where y_{t+k} = rainfall forecast at location Y for time t+k

x_t = observed rainfall at location X at time t

k = estimated lead time parameter (same as lag).

For the early stages of a storm, RAFORT uses a predetermined estimate of a, b and k based on previous analysis of the historical rainfall records. When the storm continues for a given minimum number of time intervals at both the independent and the dependent raingages, a real time updating of parameters is performed. The new values for the three parameters a, b and k are derived in two stages as follows:

1. A duoseriocorrelation (cross-correlation) analysis of the rainfall values at the two raingages X and Y is carried out. The lag time k which gives the best duoseriocorrelation between the rainfall at the two raingages X and Y is computed for every time interval using all or part of the past data at X and Y gages for the current storm. The lag value k which gives the highest duoseriocorrelation coefficient is retained for Step 2. This value of k corresponds approximately to the travel time of the storm from the independent raingage to the dependent raingage where the forecast is desired.

2. The second step involves the estimation (and updating) of the parameters a and b. With a selected value of k, in Step 1 above, the data points for the current storm events for the two raingages X and Y are fixed such that the data for Y is lagged by k time intervals with respect to the data for the independent gage X. A linear regression

analysis is then performed on all or part of the data for estimating the value of the regression constants a and b. The forecast, thereafter, is obtained from Equation 1. Use of the six to eight most recent data points in the correlation and regression calculations yielded the best results.

When more than one distant raingage is available in the same general direction of storm approach, more than one forecast could be derived for the same event. These forecasts can be given different or equal weights and integrated into one forecast. RAFORT is capable of integrating two such forecasts.

Base-Line Parameter Estimation

Estimates of the base-line parameters a, b, and k for use in the beginning stages of a real time rainfall prediction were derived by linear regression of historic rainfall at the local gage Y and rainfall at the remote gage X. An origin regression was adopted, implying a zero base-line value for 'a'. For this purpose it was found that rainfall data on a total storm basis would suffice.

The base-line value of k may be estimated by choosing that value of k which produces the highest duoseriocorrelation coefficient most of the time. In estimating the base-line value of k, hourly rainfall could be used either as a discrete hourly input or in the form of hourly values of cumulative rainfall at each gage in the pair. The latter is advisable because it produces better and more stable correlations. k may also be estimated by visual inspection of mass hyetographs of several storms for the local and the remote raingages.

A separate estimate of base-line values must be derived for each remote-local raingage pair.

Prediction Algorithms

There are four possible algorithms which the model can use for making a prediction. The basic algorithm is premised on the concept and theory described earlier. The other three algorithms were included in RAFORT to enable prediction under conditions for which the basic algorithm cannot be applied. The flowchart for RAFORT presented in Figure VI-4 shows the main decision points leading to the different predicting algorithms.

The choice of algorithm depends on different combinations of the following conditions:

a. Which raingage(s), local and/or remote are the first to report rain.

b. Whether both local and remote raingages are functioning properly.

c. The number of hours that have elapsed since the beginning of rain.

Sample Prediction

A sample prediction is shown in Figure VI-5 in two forms. Both cumulative rainfall and hourly intensities are depicted. The predicted rainfall is plotted k hours after the prediction was made. Whenever $k > 1$, a linear interpolation was used to allocate the predicted rainfall to individual hours.

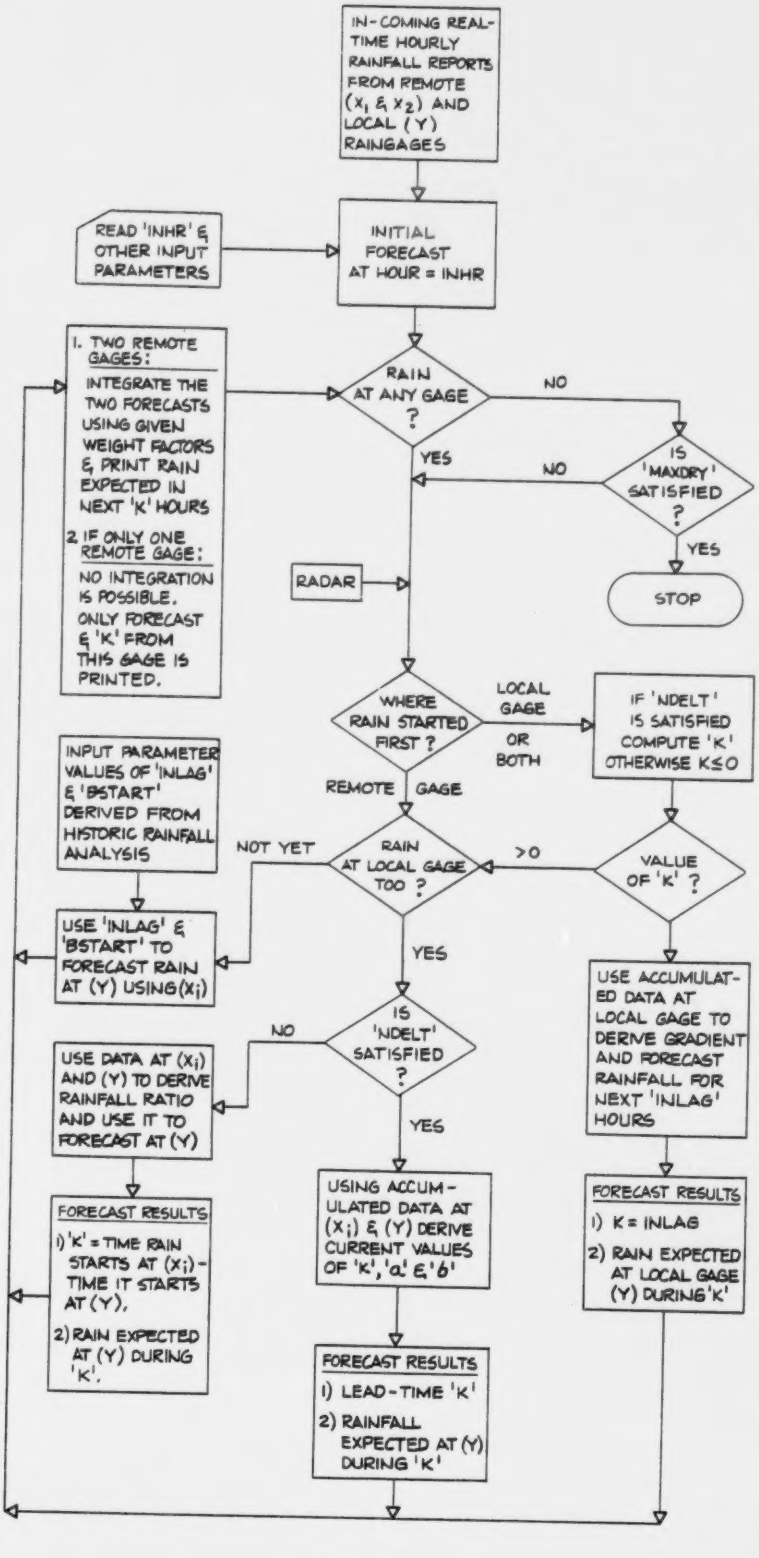

IN-COMING REAL-TIME HOURLY RAINFALL REPORTS FROM REMOTE (x_1 & x_2) AND LOCAL (Y) RAINGAGES
READ 'INHR' & OTHER INPUT PARAMETERS
INITIAL FORECAST AT HOUR = INHR
1. TWO REMOTE GAGES: INTEGRATE THE TWO FORECASTS USING GIVEN WEIGHT FACTORS & PRINT RAIN EXPECTED IN NEXT 'K' HOURS
2. IF ONLY ONE REMOTE GAGE: NO INTEGRATION IS POSSIBLE. ONLY FORECAST & 'K' FROM THIS GAGE IS PRINTED.
RAIN AT ANY GAGE ?
NO
YES
IS 'MAXDRY' SATISFIED ?
NO
YES
STOP
RADAR
WHERE RAIN STARTED FIRST ?
LOCAL GAGE
OR BOTH
REMOTE GAGE
IF 'NDELT' IS SATISFIED COMPUTE 'K' OTHERWISE K≤0
INPUT PARAMETER VALUES OF 'INLAG' & 'BSTART' DERIVED FROM HISTORIC RAINFALL ANALYSIS
NOT YET
RAIN AT LOCAL GAGE TOO ?
>0
VALUE OF 'K' ?
USE 'INLAG' & 'BSTART' TO FORECAST RAIN AT (Y) USING (x_i)
YES
IS 'NDELT' SATISFIED ?
NO
YES
USE DATA AT (x_i) AND (Y) TO DERIVE RAINFALL RATIO AND USE IT TO FORECAST AT (Y)
USE ACCUMULATED DATA AT LOCAL GAGE TO DERIVE GRADIENT AND FORECAST RAINFALL FOR NEXT 'INLAG' HOURS
USING ACCUMULATED DATA AT (x_i) & (Y) DERIVE CURRENT VALUES OF 'K', 'a' & 'b'
FORECAST RESULTS
1) 'K' = TIME RAIN STARTS AT (x_i) - TIME IT STARTS AT (Y).
2) RAIN EXPECTED AT (Y) DURING 'K'.
FORECAST RESULTS
1) K = INLAG
2) RAIN EXPECTED AT LOCAL GAGE (Y) DURING 'K'
FORECAST RESULTS
1) LEAD-TIME 'K'
2) RAINFALL EXPECTED AT (Y) DURING 'K'

FIGURE VI-4

FLOWCHART FOR RAFORT

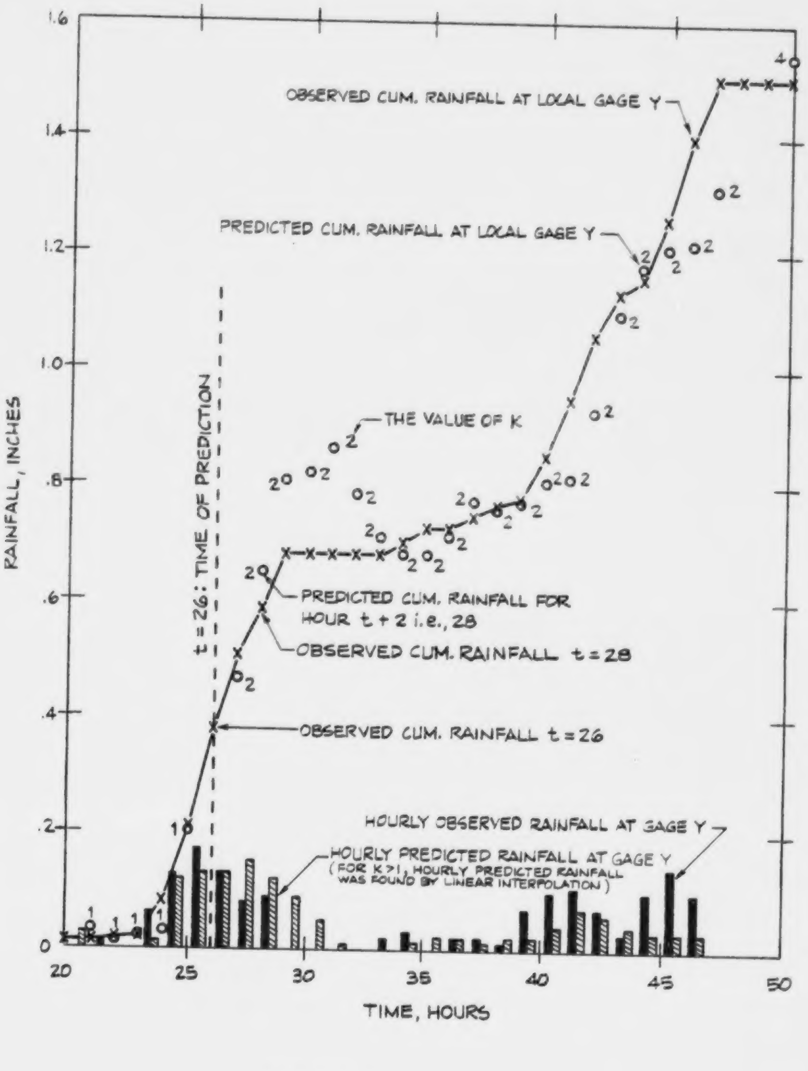


FIGURE VI-5

PREDICTED AND OBSERVED RAINFALL USING RAFORT FOR THE STORM OF NOV. 20-22, 1978. THE TIME COUNT STARTS ON HOUR 01, NOV. 20.

The forecast is performed at the end of every clock-hour (t) making use of local and remote-gage rainfall data until the current hour. A value of k is attached to each forecast since the predicted cumulative rainfall is expected to materialize at hour t + k.

Example: At t = 26 on Figure VI-5
observed cum. rainfall at local gage = 0.38 in.
estimated value of k = 2
t + k = 26 + 2 = 28
predicted cum. rainfall for hour 28 = 0.65 in.
expected rainfall from hour 26 to hour 28 = 0.27 in.

The storm used in this example occurred during November 20-22, 1978, and the rainfall data came from Tomales FD, Novato FD and FOB, the first two being the remote gages in the City's network. RAFORT derived two forecasts by using each remote gage independently with FOB and then applied equal weight factors to integrate the two forecasts. The resulting prediction is better than the average forecast made by RAFORT.

Radar Input Possibilities

RAFORT is designed such that radar-derived information expressed in an appropriate form can be easily fed into the program to be used in conjunction with remote and local rainfall data. The form of the radar input has not been pursued in depth, but it is expected to incorporate information on:

- Speed and direction of storm movement, an indication of the storm arrival time at the area of interest, and the value of k,

- Shape and orientation of storm bands, duration of approaching band and dry period between the two closest consecutive bands. In addition to its importance in the prediction model, this information constitutes useful operational input data.

- Whether the approaching storm is intensifying or dissipating, an operational input.

- The relative density of a storm at the remote gages and the point in the storm expected to hit the local gages. This is expected to improve the quality of the correlations between remote and local rainfall to be used in the rainfall prediction algorithms.

The above information when used in conjunction with real time data from the remote and local gages is expected to produce more accurate forecasts than the ones now possible.

Other Specialized Versions of RAFORT

RAFORT78

RAFORT78 uses the same prediction algorithms as RAFORT but works only with historic rainfall and accepts hourly data on a storm by storm basis to produce rainfall forecasts every hour or every 'k' hours. One of two kinds of output may be requested.

1. Within-storm detailed output which shows observed and predicted values of rainfall each prediction time, and

2. storm summary output (Table VI-1) which shows the standard error of prediction at the end of each storm for each separate forecast and for the integrated forecast.

TABLE VI-1

SAMPLE PRINTOUT FROM RAFORT78 SHOWING THE STANDARD ERROR OF PREDICTION FOR THE TWO SEPARATE FORECASTS (COLUMNS 5 & 6) AND FOR THE INTEGRATED FORECAST (COLUMN 7) FOR EACH STORM

STORM NUMBER	RAINFALL (IN) OBSERVED AT			STANDARD ERROR OF PREDICTION (%)		
	GAGEX1	GAGEX2	GAGE Y	GAGEX1	GAGEX2	INTEGRATED
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	1.84	3.33	3.13	11.7	6.8	7.4
2	0.92	2.04	1.79	15.1	23.5	16.8
3	0.57	1.33	1.63	17.5	23.5	18.2
4	1.64	2.60	2.56	10.4	14.9	10.7
5	3.28	2.93	1.97	4.9	6.3	4.5
6	4.40	4.10	1.94	16.5	11.1	12.0
7	1.38	2.00	1.30	18.2	17.9	14.7
8	0.72	1.00	0.54	52.5	48.9	41.0
9	0.75	0.80	0.50	50.7	44.6	38.7
10	2.12	3.15	2.76	7.5	6.2	5.7
11	3.01	3.23	1.70	15.6	11.2	11.0
12	0.69	1.19	0.51	36.0	27.4	22.1
13	0.49	0.70	0.43	26.6	36.7	27.1
14	1.13	1.25	1.48	26.5	30.9	26.1
15	1.07	1.07	0.23	51.2	36.6	38.3
16	1.09	1.10	0.50	59.6	42.3	41.3
17	0.98	1.10	0.56	102.2	86.9	61.3
18	0.70	0.70	0.74	16.3	13.4	10.7
19	0.85	0.80	0.61	38.9	113.3	88.6
20	1.08	1.68	0.79	12.3	17.5	10.2
21	1.77	2.10	0.62	35.0	25.3	29.3
22	2.80	2.86	1.44	10.3	7.2	6.9
23	2.25	2.45	0.76	40.6	49.2	36.5
24	0.70	0.71	0.66	26.5	17.5	18.7
25	1.10	1.83	0.89	11.3	8.8	8.6
26	0.12	0.38	0.87	66.5	52.5	54.7
27	0.97	1.18	0.59	42.5	27.2	32.7
MEAN STANDARD ERROR OF PREDICTION				30.5%	29.9%	25.7%

GAGEX1 = TOMALES FS
WT FACTOR = 1.0

GAGEX2 = NOVATO 8WNW
WT FACTOR = 1.0

GAGE Y = FOB IN SF

The second output enables quick evaluation of the merits of an integrated forecast from two remote gages versus the forecast from one remote gage only. (It was found, for a number of storms, that the integrated forecast gave a lower standard error of prediction). This output also enables a check on the accuracy of the predictions for storms of different magnitude (Figure VI-6).

An additional feature of RAFORT78 is its capability to calculate the absolute error of prediction whenever the observed rainfall at the FOB gage exceeds a given base value. The error occurrences in each given magnitude class are counted and printed out in tabular form. Figure VI-7 shows the results for the 27 storms of Table VI-1 in the form of a probability distribution plot of the computer output.

RAFORT78 may also be used with historic data and the standard error criterion to test and optimize values of certain input parameters or variables.

RAFORT19

RAFORT19 is a slightly modified version of the rainfall prediction model RAFORT and uses the same prediction algorithms as used in the predictive reconstruction of 28 years of rainfall at the FOB gage in San Francisco. The predicted rainfall file covers 28 hydrological years, 1948/49 - 1975/76. Every hour a rainfall value is predicted for the following 'k' hours, $1 < k < 5$. The predicted rainfall depth is a lumped value expected during the 'k' hours and is not alloted to individual hours during 'k'.

The independent (remote) raingage used in the prediction is Novato 8WNW and the local station is the FOB raingage in San Francisco. The 28-year rainfall record at the remote gage has several data gaps and the resulting predicted rainfall file is not up to the accuracy standards achievable under today's conditions. Nevertheless, the latter has been used as the predicted

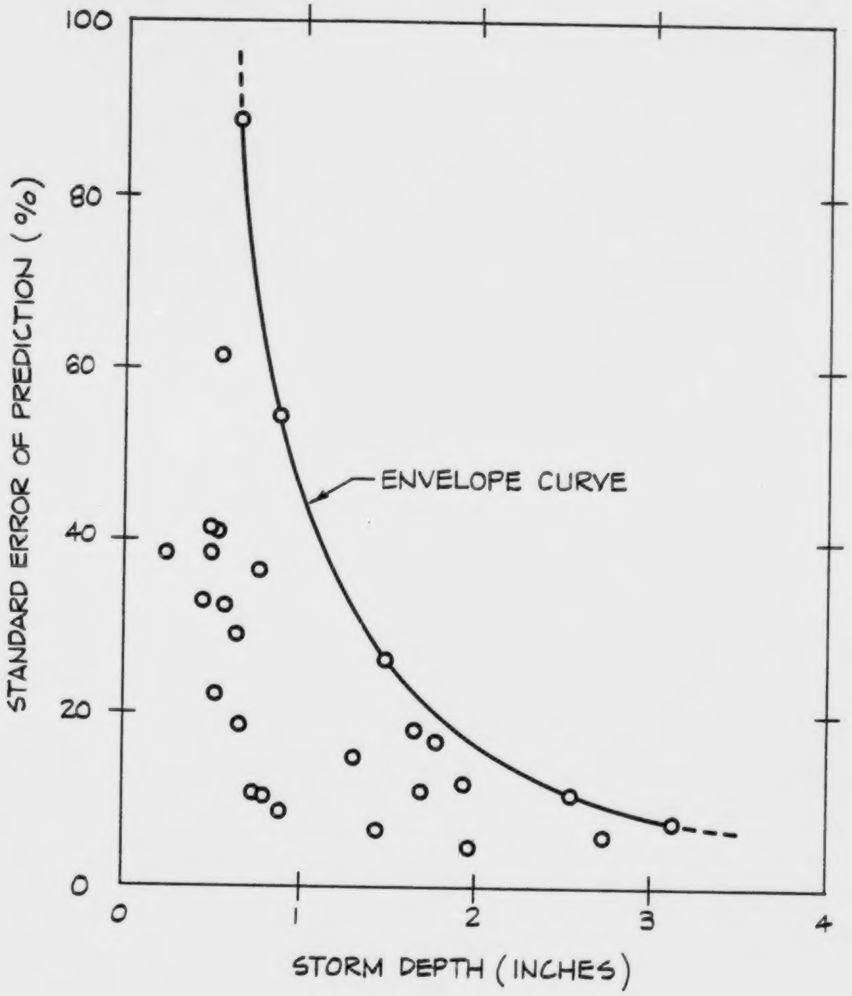


FIGURE VI-6

STANDARD ERROR OF PREDICTION VS. STORM DEPTH.
THE DATA USED IS FROM COLUMNS 4 AND 7 OF TABLE 1

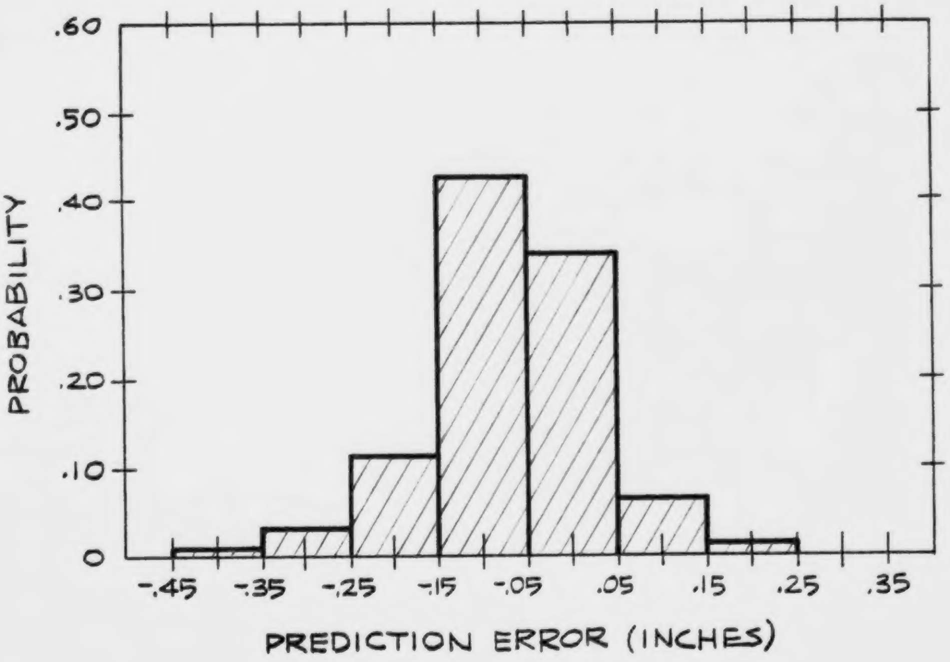


FIGURE VI-7

PROBABILITY DISTRIBUTION OF PREDICTION ERROR FOR THE 27 STORMS IN TABLE 1. HOURLY RAINFALL AT FOB LESS THAN 0.08 IN. WAS NOT COUNTED.

rainfall input for use in evaluating predictive-reactive real time control strategies of the program TREAT. The FOB (observed) record was used as a "prefect prediction" to determine the potential gains that could be made by a predictive-reactive strategy.

DISCUSSION

By making use of current rainfall observations at distant locations and raingages in the area of interest, together with base-line information extracted from historic rainfall data, real time rainfall prediction seems to be feasible. Both rainfall prediction models, RAFORT (described herein), and FORCST (developed by CSU), claim encouraging results.

Of course, the accuracy of rainfall predictions of this kind is constrained by the degree of randomness woven into the rainfall mechanism, by the structure and movement of a storm system and by the availability of reliable real time rainfall data from points in the direction of storm approach and also from local raingages. For the specific case of San Francisco some of these limitations are valid, singularly or collectively. The accuracy of test predictions performed on a number of storms ranged from low to extremely good. A relative measure of prediction accuracy is the standard error of prediction which is given in Table VI-1 for 27 storms. The last column of this table lists the results for the integrated prediction. The plot of observed rainfall depths versus the standard error of prediction in Figure VI-6 indicates that the accuracy of prediction as expressed by the standard error improves with increasing storm rainfall depths.

A presentation of the absolute prediction error for the 27 storms of Table VI-1 is given in Figure VI-7. Storms less than 0.50 inches (12.7 mm) were left out and hours during which the FOB gage received less than 0.08 inches (2 mm) of rain were not considered in the error count. The error probability plot in Figure VI-7 shows a more frequent occurrence of negative error, indicative of underprediction.

Preliminary tests have suggested that further improvements of the quality of rainfall predictions with RAFORT may be possible by using a mean rainfall value from a group of local gages instead of rainfall from a single point as used so far. For the same storms listed in Table VI-1, the average Standard Error of Prediction when using areal rainfall values as the local gage input is 24.1 percent as compared to 25.7 percent with point rainfall inputs. The spatial lumping obviously dampens the effects of the high spatial variation observed in short-duration rainfall.

RAFORT does not provide for spatial allocation of predicted rainfall, but rather gives the rainfall expected to occur during the period covered by the prediction (k hours) in the vicinity of the local or dependent raingage(s). For areas larger than San Francisco (45 sq. miles or 116 sq. km), rainfall predictions may be derived for more than one location provided that rainfall data from these locations is available. The CSU model FORCST is built to accept inputs from a number of different locations and to issue rainfall forecasts for these locations. A combination of RAFORT and FORCST, the first with the remote - local rainfall correlation concept and the second with the spatially distributed rainfall forecast capability, may be worth investigating.

VII. EVALUATION OF ALTERNATIVE CONTROL STRATEGIES

GENERAL APPROACH

Five types of control strategies were studied for the North Shore System. These strategies were discussed in Chapter IV with regard to control philosophy (reactive or integrated reactive-predictive), control method (manual or automatic), and the types and location of control devices and computer hardware and software required to implement the strategies. In this chapter, the five strategies are further defined, and the results of performance and cost studies are presented. The approach used in studying alternative control strategies consisted of two preliminary steps:

1. Formulation of general categories of alternative control strategies.
2. Selection of performance indicators.

Each of the five general control strategies selected for further study was then assessed using the following steps:

1. Modification of the TREAT computer model (see Chapter V) to allow it to simulate the control strategy.
2. Formulation of one or more specific control sub-strategies within the broad strategy being studied.
3. Long-term simulation (usually 70 years) of the NSOC system performance using historical rainfall data and ultimate dry weather flows.
4. Reduction and analysis of TREAT output.
5. Formulation, simulation and analysis of additional specific sub-strategies, as determined by previous TREAT results.

Following the analysis of all five general control strategies in this manner, the cost of the best sub-strategy within each general strategy was determined, including capital and O&M costs. Based on the performance assessment and the cost comparisons, a cost-effectiveness analysis was performed.

PERFORMANCE INDICATORS

To compare the performance of the alternative control strategies, twelve performance indicators were selected. The TREAT model was programmed to produce a complete statistical frequency distribution for each indicator. The selection of the twelve indicators was based on consideration of earlier TREAT runs which characterized the NSOC system and suggested several measures that would be useful and significant indicators of performance.

The twelve performance indicators selected can be divided into three categories based on whether they measure the number, amount or duration of the performance indicator. The twelve indicators are listed and briefly defined below.

Number of Events

- Use of Primary Treatment - the number of times per year the North Point Pump Station and Water Pollution Control Plant would be placed in operation to handle wet weather flows. The primary treatment event is over when the inflow rate to the transport-storage system can be handled by the dry weather pumps and storage is empty.
- Use of Storage - the number of times per year storage would be used. Each storage event is considered over when the storage facilities are emptied.
- Untreated Overflows - the number of times per year the storage capacity of the system would be exceeded, resulting in the discharge of untreated waste water.*

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*Limiting overflows to an average of one per year had been the primary objective of the NSOC system, as dictated by the NPDES requirement of the California Regional Water Quality Control Board. This requirement was the basis for the sizing of the transport-storage system and was considered a fixed constraint throughout the RTAC study. However, the City of San Francisco has recently successfully applied to the Regional Board to have the number of allowable overflows increased to an average of 4 per year.

Annual Amount of Events

- Secondary Treatment (MG/yr) - the annual amount of wastewater (dry weather and wet weather flows) that would receive secondary treatment at the Southeast Treatment Plant.

- Primary Treatment (MG/yr) - the annual amount of wastewater that would receive primary treatment (North Point or Southwest Plant).

- Untreated Overflows (MG/yr) - the annual amount of wastewater that would overflow untreated to San Francisco Bay.

- Maximum Storage Level (MG) - the annual maximum amount of wastewater in storage in the NSOC transport-storage system.

Duration of Events

- Primary Treatment Season (Days/yr) - the annual number of days from the first use of primary treatment facilities to the last use of these facilities; a year begins on July 1 and ends on June 30.

- Storage Season (Days/yr) - the annual number of days from the first use of the storage facilities to the last use of these facilities; a year begins on July 1 and ends on June 30. Storage occurs whenever the available treatment capacity is exceeded.

- Primary Treatment Events (Hrs/yr) - the total annual number of hours the primary treatment facilities would be in operation. This time does not include start-up or shut-down time.

- Storage Events (Hrs/yr) - the total annual number of hours the storage facilities would contain wastewater that is being held for later treatment (or overflow).

- Untreated Overflows (Hrs/yr) - the total annual number of hours in which untreated overflows would occur.

Pollutant Reduction Factor

In addition to the twelve basic performance indicators listed above, a rough measure of the reduction in pollutant load discharged was computed for each strategy, using the average annual amounts of wastewater receiving primary, secondary, or no treatment (overflow). A rigorous computation of such a measure would need to consider each pollutant separately and the changes in pollutant concentrations due to sedimentation and scour in the storage and transport facilities, BOD decay and coliform dieaway, and the time varying concentrations of each pollutant from dry and wet weather inflow. Such a complex analysis was not appropriate for the RTAC study. Instead, certain simplifying assumptions were made and a pollutant reduction factor was formulated as follows:

1. The reductions were computed in comparison to the Basic Operating Strategy (Alternative I). The amounts of flow receiving primary, secondary, or no treatment (overflow) were taken from the TREAT run for this strategy.
2. Based on typical measured values for the concentrations of BOD in dry weather and wet weather flows, a ratio of ten to one was assumed to be reasonable (dry weather concentrations, 200-300 mg/l and wet weather concentrations, 20-30 mg/l).

3. Because the operating rule for Strategy I is to shut down the dry weather pumps as soon as any significant wet weather flows occur (thus leaving the full capacity of the Southeast Waste Water Treatment Plant available for wet weather flows in the southeast section of the City), it may be assumed that all of the flow receiving secondary treatment in this case is dry weather flow. The remainder of the dry weather flow is that which occurs during storm periods and it receives primary treatment. In the North Shore area all wet weather flows, except the overflows, receive primary treatment.
4. The mass emissions of pollutants were computed by assuming the ten to one ratio of dry versus wet weather flow concentrations and a pollutant removal rate of 90 percent for secondary treatment and 40 percent for primary treatment.
5. The mass emissions for all other strategies were also computed, based on the TREAT results for the amount of flow receiving primary, secondary, and no treatment. To take into account that more dry weather flow receives secondary treatment when the wet weather pumps are not activated as soon as under the Basic Operating Strategy, the duration of primary treatment for each case was compared to the duration for Alternative I and the amount of dry weather flow receiving primary treatment was assumed to be that ratio times the amount receiving primary treatment in Alternative I. The remainder of the dry weather flow was assumed to receive secondary treatment.
6. The reduction in mass emissions (or pollutant load) for each alternative strategy was then expressed as a percent reduction from the load for Alternative Strategy I.

ALTERNATIVE I - BASIC OPERATING STRATEGY

Introduction

The basic operating strategy consists of one simple rule: the North Point Pump Station and Water Pollution Control Plant should always be brought on-line as quickly as possible and be operated at the highest possible rate during wet weather periods. This reactive strategy minimizes the number of overflows from the system.

Simulation Results

The results of the TREAT run for the basic operating strategy are shown in Table VII-1. For each of the twelve performance indicators, the minimum, average and maximum annual values are presented. Several important observations were made from these findings with regard to overflows, storage and treatment. The most significant findings are discussed below.

Overflows

The average number of overflows per year is expected to be 0.86, which satisfies the one overflow per year requirement. Over the 70 years simulated, the number of overflows per year varied from 0 to 4. The breakdown in percentage of years in which a certain number of overflows is expected to occur is given below.

Number of Overflows in a Year	Percent of Years Having the Given Number of Overflows
0	45
1	27
2	24
3	2
4+	2

As shown above, in almost half the years no overflows occur. Referring to Table VII-1, the average volume of untreated overflows is 8.9 MG, which represents only about one-tenth of one percent of the total combined wastewater. In 80 percent of the years, the annual overflow is less than 17 MG, or less than two-tenths of one percent of the total wastewater. Also, the annual duration of overflows ranges from 0 to 10 hours, with an average of only 1.2 hours. One can conclude from these results that untreated overflows are well controlled under the Basic Operating Strategy.

TABLE VII-1

SUMMARY OF RESULTS FOR ALTERNATIVE STRATEGY I
BASIC OPERATING STRATEGY

Performance Indicator	Minimum Event	Average Event	Maximum Event
Number of Events			
Use of Primary Treatment	50	106	190
Use of Storage	8	23	49
Untreated Overflows	0	0.86	4
Annual Amount of Events			
Secondary Treatment (MG)	7100	7311	7477
Primary Treatment (MG)	446	1196	2106
Untreated Overflows (MG)	0	8.9	69
Maximum Storage Level (MG)	8.8	24.0	27.4
Duration of Events			
Treatment Season (Days)	146	258	352
Storage Season (Days)	59	190	300
Primary Treatment Events (Hrs)	147	328	570
Storage Events (Hrs)	21	86	194
Untreated Overflows (Hrs)	0	1.2	10
Reduction in Pollutant Load (%)		-	

Figure VII-1 is a plot of the maximum annual overflow rate (cfs) versus frequency of occurrence. Also shown are the capacities of five outfalls that are to be given priority for overflows. It can be seen that the five outfalls should be adequate.*

Storage

Table VII-1 indicates that the number of times storage is utilized in an average year is only 23, as compared to 106 uses of the primary treatment facilities. The results for the duration of storage use are also small relative to the duration of primary treatment usage. The average storage season is only 190 days long. In addition, we found that in almost half the years, no overflows occur, so we know that storage is not fully utilized in those years. These findings all suggest that there is a large amount of seldom used storage capacity in the system which could be better taken advantage of to allow more of the wastewater to receive secondary treatment. When storage does occur, the results tell us that the average duration of a storage event is 3.7 hours (using average values for the number of events and total duration of storage).

Treatment

The average annual amount receiving secondary treatment (7311 MG) is less than the average annual dry weather flow (7572 MG); this occurs because the wet weather pumps would come on-line as soon as wet weather flows are present and divert portions of the dry weather flow to the primary treatment plant. About 86 percent of the total wastewater receives secondary treatment. As expected, the amount of flow receiving primary

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*Subsequent analysis revealed that this is also true for all alternative strategies. The strategies differ only in the frequency of use of the initial outfalls.

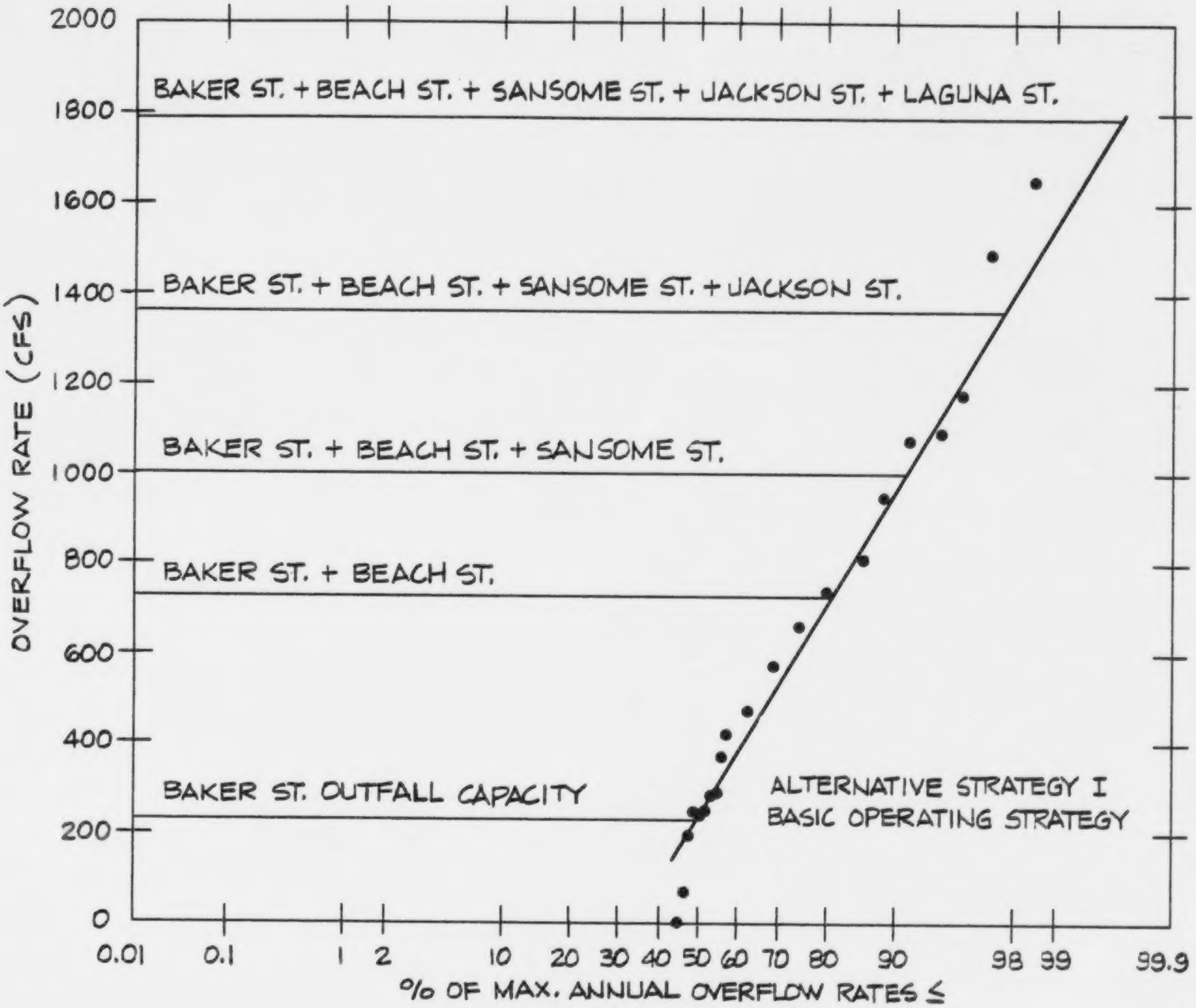


FIGURE VII-1

MAXIMUM ANNUAL OVERFLOW RATE: FREQUENCIES AND OUTFALL CAPACITIES FOR ALTERNATIVE STRATEGY I - BASIC OPERATING STRATEGY treatment would vary much more widely from year to year than the amount receiving secondary treatment, but it would average about 14 percent of the total wastewater flow. Primary treatment facilities would be used an average of 106 times a year for an average of about 3 hours and would treat an average of 11 MG each time the facilities would be placed in operation (based on average values for the number of events and total duration and amount of events). The length of the primary treatment season is highly variable, reaching a maximum of nearly all year (352 days). Note, however, that the average season is only 258 days or about 8 1/2 months long. This suggests that the performance of the system may not be affected very much by suspending the operation of the primary treatment plant for part of the year.

The pollutant load reduction is zero since this strategy is the base case to which all other strategies will be compared for estimating pollutant reduction. Pollutant reduction may be increased by increasing the amount of secondary treatment and decreasing the amount of primary treatment. This can only be done by utilizing storage more effectively, which also implies a greater risk of overflows that could negate the gain. This effect is further explored by subsequent strategies.

To summarize, the Basic Operating Strategy controls overflows effectively, limiting them to less than one per year, but results in frequent use of the primary (or wet weather) treatment facilities for short time periods and poor utilization of storage. There are also indications that year-long operation of the primary treatment facilities may not be required to meet the system objectives.

ALTERNATIVE II - BASIC OPERATING STRATEGY WITH SEASONAL CONSTRAINTS

Introduction

The Basic Operating Strategy requires that the North Point WPCP and Pump Station be available for use at all times. Analysis of this strategy showed that although the maximum length of season for use of the wet weather plant was 352 days, the average season length was only 258 days. In view of this and the fact that most of the large rainstorms occur in the same seven months of the year, it is reasonable to expect that the operation of the plant could be limited to that portion of the year without significant adverse impacts on performance of the North Shore Outfall Consolidation (NSOC) Project. This led to the development of seasonal constraint strategies.

Development of Alternative Strategies

To determine which months the North Point WPCP should be in operation, a monthly distribution of overflow events was developed from TREAT runs under the Basic Operating Strategy. Figure VII-2 shows these results; the figure shows there is little risk of overflows during the months of May, June, July, August, and September. Furthermore, the possibility of overflows in October and April is relatively low compared to the November through March period. These results, though not unexpected, provide useful information for the purpose of developing effective operating strategies.

Based upon this distribution of overflow events, four alternative strategies were developed. These strategies differ in the months when the wet weather treatment facilities are assumed to be operational. The four strategies and their corresponding wet weather treatment seasons are:

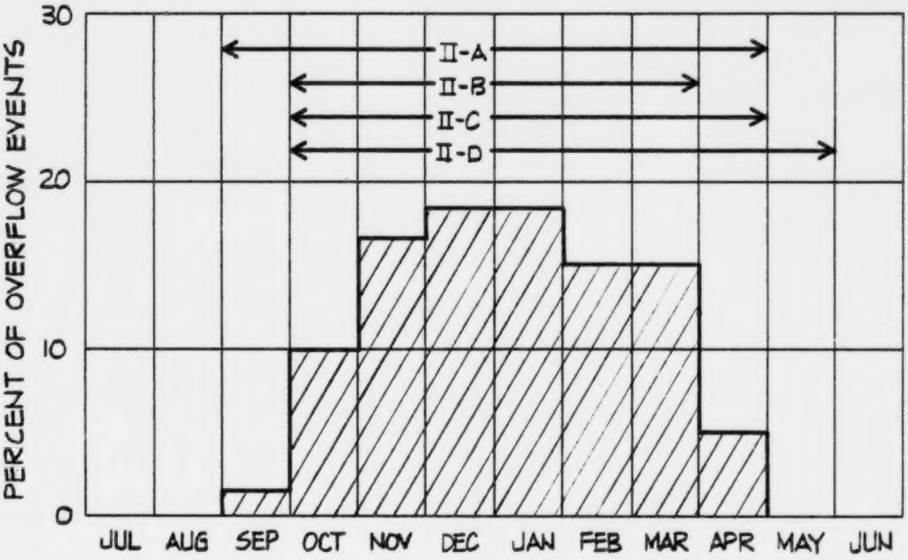


FIGURE VII-2

MONTHLY DISTRIBUTION OF OVERFLOW
EVENTS AS SIMULATED BY TREAT MODEL
FOR THE BASIC OPERATING STRATEGY

Strategy	Wet Weather Season
II-A	September 1 - April 30
II-B	October 1 - March 31
II-C	October 1 - April 30
II-D	October 1 - May 31

These four seasons are illustrated in Figure VII-2.

Simulation Results

Table VII-2 summarizes the average annual values obtained from TREAT runs for all four sub-strategies along with the results for the Basic Operating Strategy. Generally speaking, the results show that in comparison to the Basic Operating Strategy, the seasonal constraint strategies:

1. Provide more wastewater with secondary treatment and less with primary treatment;
2. decrease the number of times the primary treatment plant must be started up, the total duration the primary treatment plant must operate, and the length of the wet weather treatment season;
3. increase the number, amount, and duration of untreated overflows;
4. increase the use of storage, as evidenced by increases in the number of storage events, total duration storage is utilized, the length of the storage season, and maximum storage levels; and
5. reduce the pollutant load discharged to the Bay.

TABLE VII-2

SUMMARY OF RESULTS FOR ALTERNATIVE STRATEGY II BASIC OPERATING STRATEGY WITH SEASONAL CONSTRAINTS (MEAN ANNUAL VALUES)

PERFORMANCE INDICATOR	ALTERNATIVE STRATEGY				
	I	II-A	II-B	II-C	II-D
Number of Events					
Use of Primary Treatment	106	97	88	97	104
Use of Storage	23	29	29	27	25
Untreated Overflows	0.86	1.1	1.4	1.1	0.93
Average Annual Amount of Events					
Secondary Treatment (MG)	7311	7338	7424	7357	7323
Primary Treatment (MG)	1196	1163	1064	1144	1183
Untreated Overflows (MG)	8.9	15	27	15	9.9
Maximum Storage Level (MG)	24.0	24.5	25.4	24.5	24.2
Duration of Events					
Treatment Season (Days)*	365	242	182	212	243
Storage Season (Days)*	300	352	352	352	352
Primary Treatment Events (Hrs)	328	327	290	314	321
Storage Events (Hrs)	86	187	297	186	106
Untreated Overflows (Hrs)	1.2	3.6	7.4	3.7	1.5
Reduction in Pollutant Load (%)	-	0.2	1.9	0.7	0.3

*Maximum Values

The magnitude of these effects is greatest for Strategy II-B, followed by Strategies II-C and II-A, with Strategy II-D showing little change from the Basic Operating Strategy.

Since the performance of the system under all four alternative strategies is identical to that of the Basic Operating Strategy in those months when the primary treatment plant is operating, the extent of the performance change is a function of how many and which months the plant is not available for use. Closing the plant for the months of June - September (Strategy II-D) has almost no effect on system performance (except, of course, for the length of the wet weather treatment season); while shutting down for April - September (Strategy II-B) has a more significant effect. The nature of the effect is favorable in every regard for all strategies except the number, amount and duration of untreated overflows. These increase from 0.86/year and 8.9 MG/year under the Basic Operating Strategy to 1.4/year and 27 MG/year for the six-month treatment season (Strategy II-B). It is notable that the increase in untreated overflows is more than made up for by the increase in secondary treatment as reflected by the net reduction in pollutant load.

Since none of the four strategies differ from the Basic Operating Strategy during the wet weather treatment season (when the great majority of events take place), performance improvements are modest. None of the strategies overcomes the major limitations of the Basic Operating Strategy: poor use of storage and frequent use of the wet weather treatment plant for short periods of time. However, the results are significant because they indicate about the same or slightly better system performance can be achieved by keeping the wet weather plant operating for only part of the year. Further operating improvements will require strategy modifications that directly affect the majority of events.

ALTERNATIVE III - BASIC OPERATING STRATEGY WITH MINIMUM STORAGE CONSTRAINTS

Introduction

The limiting features of the Basic Operating Strategy are 1) it does not utilize the available storage very effectively, and 2) it places a high degree of dependence on the wet weather treatment plant, using it frequently for short periods of time. These limitations occur because the Basic Operating Strategy assumes the wet weather treatment plant will be put into operation as soon as flow into the system exceeds the maximum dry weather flow capacity. This condition occurs essentially every time it rains.

Many storms would not fill the available storage even if the wet weather pump station and treatment plant were never used. In these cases, the stored water could be treated at the secondary treatment plant (Southeast), thus eliminating the need to operate the wet weather primary treatment plant for short periods of time and increasing the removal of pollutants. On the other hand, there are storms that will eventually exceed the system's storage capacity and result in untreated overflows. Not activating the wet weather treatment plant immediately in these cases could result in larger overflows. In the case of other events, delaying the start-up of the wet weather treatment plant could result in overflows that would not have otherwise occurred. The ideal strategy would avoid starting up the wet weather treatment plant for the many small storms which would not cause an overflow, yet start the plant up soon enough in the other cases to keep overflow volumes to a minimum and avoid most of the unnecessary overflows.

Short of using rainfall prediction, one strategy would be to wait until the amount of wastewater in storage reaches a predetermined minimum level before the wet weather pumps and treatment plant are activated. The longer the wait, the more wastewater that can receive secondary treatment and the greater the risk of untreated overflows. The best minimum storage level can be determined by evaluating the system's response (using TREAT) under different operating rules.

Development of Alternative Strategies

Nine strategies were formulated and evaluated, each differing only in the minimum storage level that must be reached before the wet weather pumps and treatment plant are activated. These minimum storage levels ranged from 3 percent to 75 percent of the available wet weather storage. The alternative strategies are described in Table VII-3.

Simulation Results

Results of the TREAT runs are summarized in Table VII-3. Average annual values of the performance indicators and the pollutant reduction percentages for each alternative as well as for the Basic Operating Strategy are given. Figures VII-3 through VII-6 illustrate selected performance measures as they vary with minimum storage level, expressed as a percentage of total storage capacity.

In general, the results for all performance indicators exhibit either a steadily increasing or decreasing behavior as the minimum storage level increases. The greatest changes tend to occur when the minimum storage level increases from 0 percent (Basic Operating Strategy) to 10 percent of the storage capacity. The notable exception to this trend is overflows (number, volume and total duration); these measures increase slowly at first and then more rapidly for the higher minimum storage levels. Since the overflow indicators are the only ones that exhibit negative improvement with increasing minimum storage levels, significant system improvement can be realized without greatly affecting overflow volume and frequencies by increasing the minimum storage slightly.

One unexpected result is that the reduction in pollutant load continues to show improvement even with the minimum storage level at 75 percent of storage capacity as shown in Figure VII-3. The reason for this becomes clearer after

TABLE VII-3

SUMMARY OF RESULTS FOR ALTERNATIVE STRATEGY III
BASIC OPERATING STRATEGY WITH MINIMUM STORAGE CONSTRAINTS
(MEAN ANNUAL VALUES)

STRATEGY	I	III-E	III-F	III-A	III-G	III-B	III-C	III-D	III-H	III-I
% CAPACITY	0	3.1	6.3	7.8	10.9	15.6	23.4	31.3	50	75
STORAGE (IN)	0.00	0.02	0.04	0.05	0.07	0.10	0.15	0.20	0.32	0.48
Number of Events										
Primary Treatment	106	68	56	53	46	40	32	27	20	14
Storage	23	99	89	86	82	76	69	64	58	51
Untreated Overflows	0.86	0.87	0.91	0.96	0.99	1.1	1.3	1.5	2.1	3.5
Average Annual Amount of Events										
Secondary (MG)	7311	7448	7494	7510	7540	7576	7626	7664	7737	7825
Primary (MG)	1196	1058	1012	996	966	929	876	837	758	657
Overflows (MG)	8.9	9.0	9.5	9.8	10.3	11.4	13.2	15.3	21.4	33.9
Maximum Storage (MG)	24.0	24.0	24.2	24.3	24.6	25.0	25.5	26.0	26.8	27.3
Duration of Event										
Treatment Season (Max. Days)*	352	338	313	313	313	313	277	277	271	258
Storage Season (Max. Days)*	300	352	352	352	352	352	352	352	352	352
Primary (Hours)	328	230	204	196	182	168	151	141	124	105
Storage (Hours)	86	264	344	370	419	480	566	630	762	920
Overflows (Hours)	1.2	1.2	1.3	1.3	1.4	1.6	1.8	2.1	2.9	4.8
Reduction in Pollutant Load (%)	-	4.4	5.7	6.0	6.7	7.4	8.4	8.9	9.9	11.0

*Maximum Value

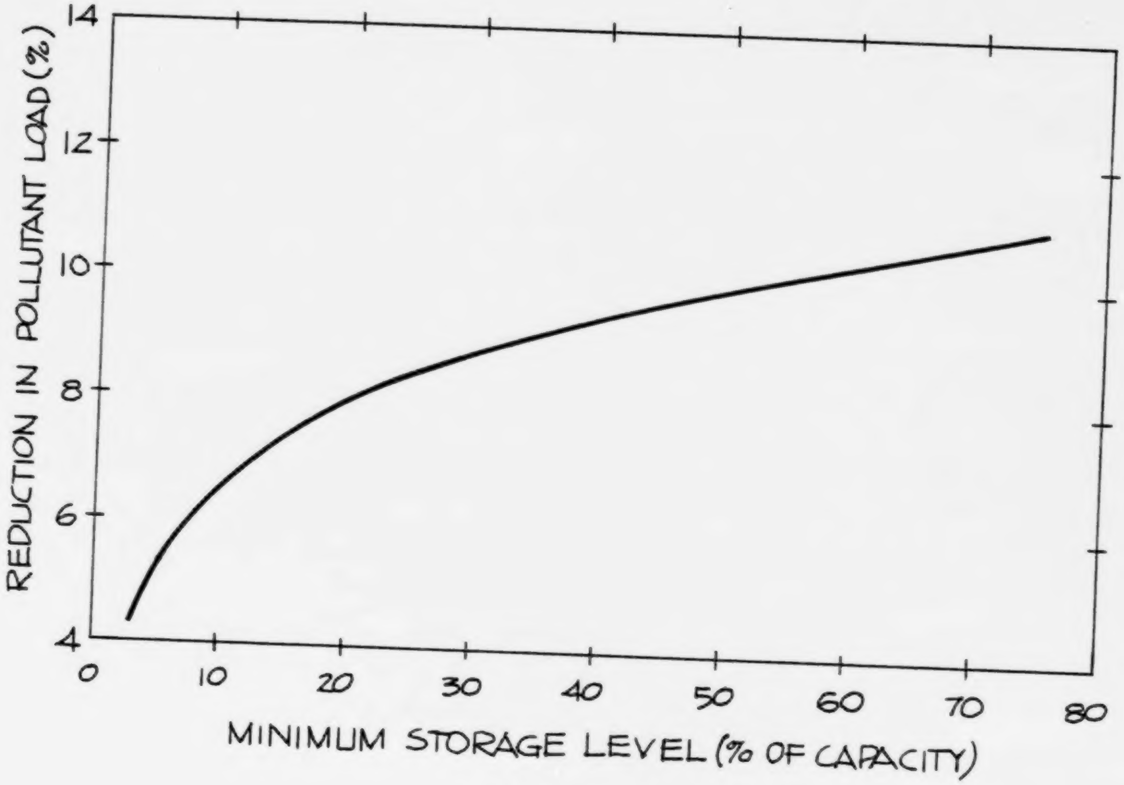


FIGURE VII-3

REDUCTION IN AVERAGE ANNUAL POLLUTANT LOAD VERSUS MINIMUM STORAGE LEVEL FOR ALTERNATIVE STRATEGY III BASIC OPERATING STRATEGY WITH MINIMUM STORAGE CONSTRAINTS inspection of Figure VII-4, which illustrates the relative changes in volume of wastewater receiving secondary treatment, primary treatment and overflowing untreated at different minimum storage levels. As can be seen, increasing the minimum storage level has a much greater effect on improving the level of treatment than on increasing overflows. The net result is a reduction in pollutant loads, even at very high minimum storage levels.

Another significant operating improvement is the dramatic decrease in the number of primary treatment events as the minimum storage level increases. (A primary treatment event is when the wet weather pump station and treatment plant must be activated). Figure VII-5 shows that a minimum storage level of only 10 percent of the storage capacity reduces the number of times the wet weather treatment plant is activated to less than half the number that would occur under the Basic Operating Strategy. This decrease in the number of primary treatment events is accompanied by a decrease in the total number of hours (duration) the primary treatment is used (Figure VII-6). Using average annual values as reported in Table VII-3, the average duration of a primary treatment event increases from 3.4 hours to 7.5 hours as the minimum storage level increases from 3 percent to 75 percent of storage capacity. For the Basic Operating Strategy, the corresponding value is 3.1 hours per event. Therefore, Strategy III results in less frequent use of primary treatment, but longer periods of time per use.

Storage is also used more effectively with this strategy. As compared to the Basic Operating Strategy, the number of storage events quadruples for the case of a 10 percent minimum storage level. The number of storage events is a maximum at a minimum storage level of 3 percent and declines gradually as what were previously considered separate storage events combine into single events. Figure VII-6 further illustrates that the use of storage increases steadily with increasing minimum storage levels as measured by the total duration of storage. These facts also point out the average duration of each storage event increases as the minimum storage level increases. Using average annual values for the number of storage events and total duration of storage,

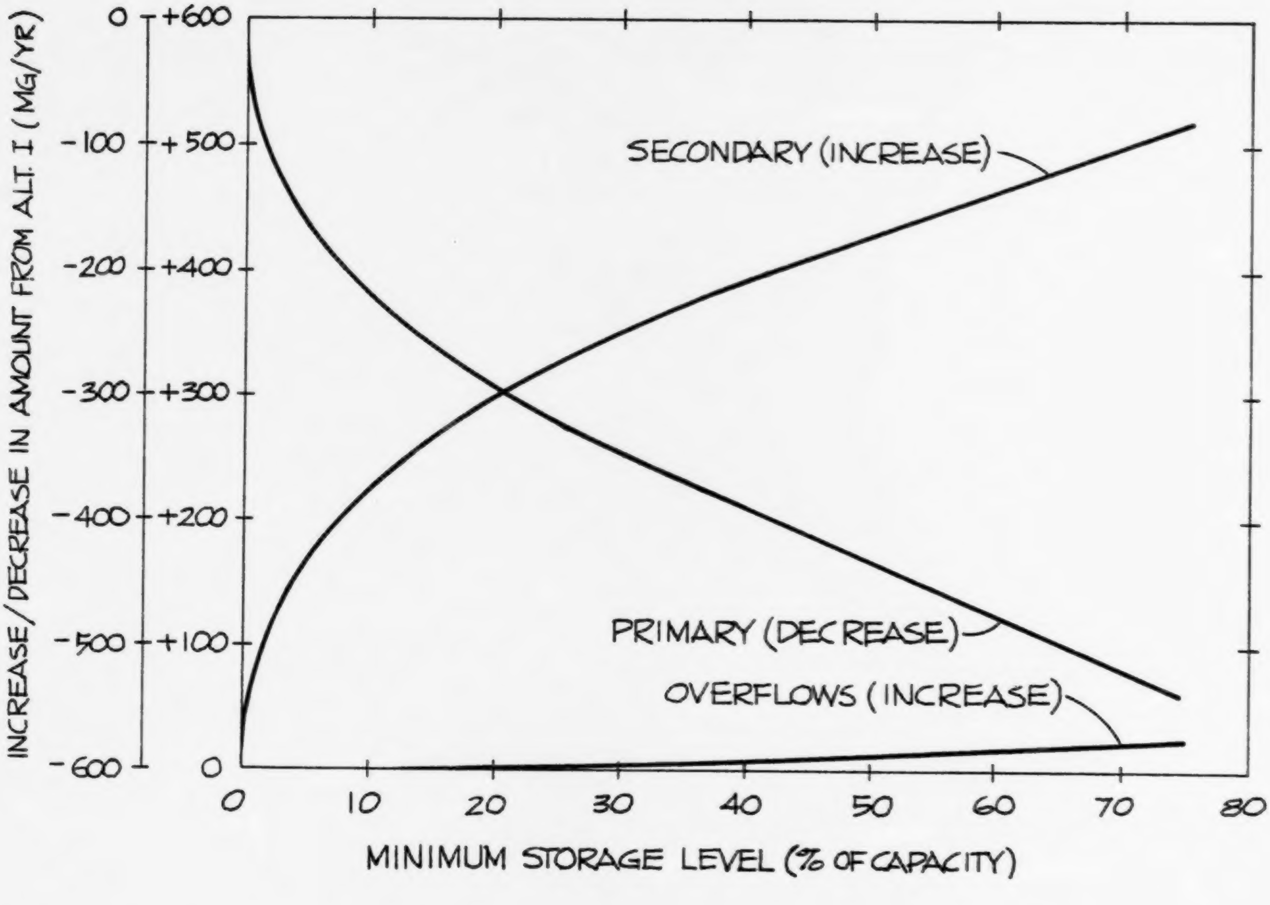


FIGURE VII-4

CHANGES IN AVERAGE ANNUAL AMOUNT OF TREATMENT AND OVERFLOWS VERSUS MINIMUM STORAGE LEVEL FOR ALTERNATIVE STRATEGY III BASIC OPERATING STRATEGY WITH MINIMUM STORAGE CONSTRAINTS

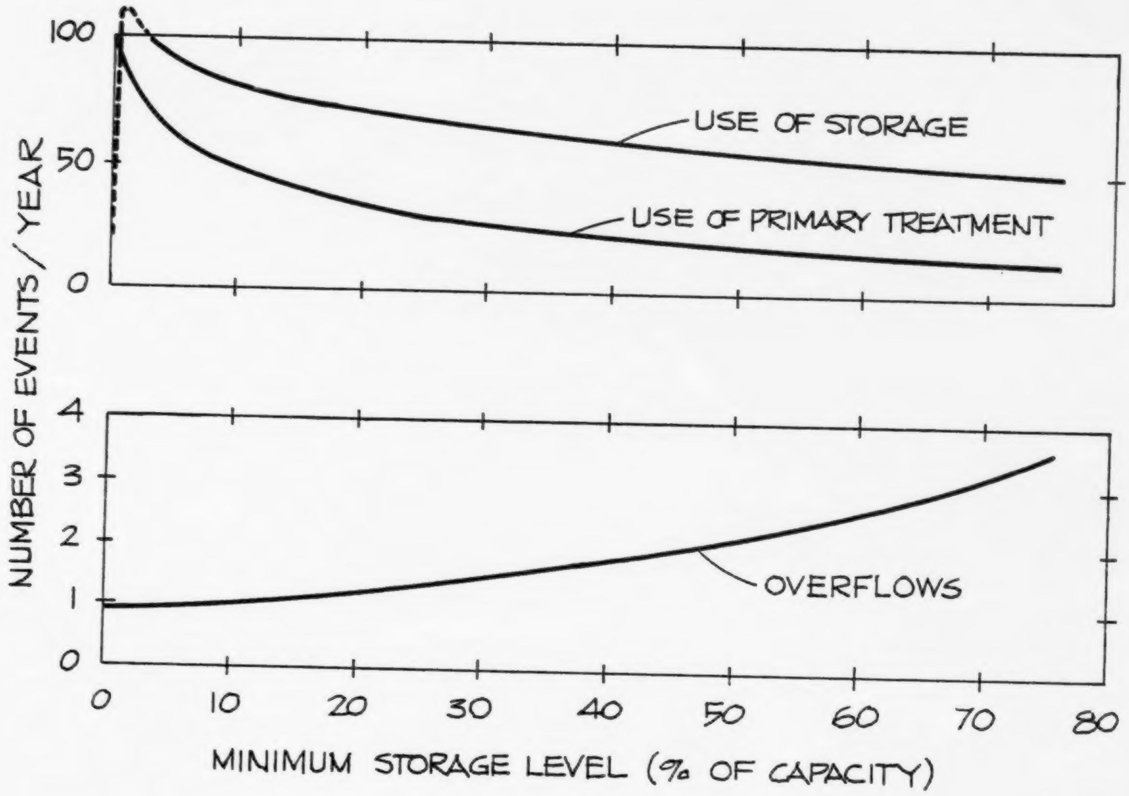


FIGURE VII-5

AVERAGE ANNUAL NUMBER OF STORAGE, TREATMENT AND OVERFLOW EVENTS VERSUS MINIMUM STORAGE LEVEL FOR ALTERNATIVE STRATEGY III BASIC OPERATING STRATEGY WITH MINIMUM STORAGE CONSTRAINTS

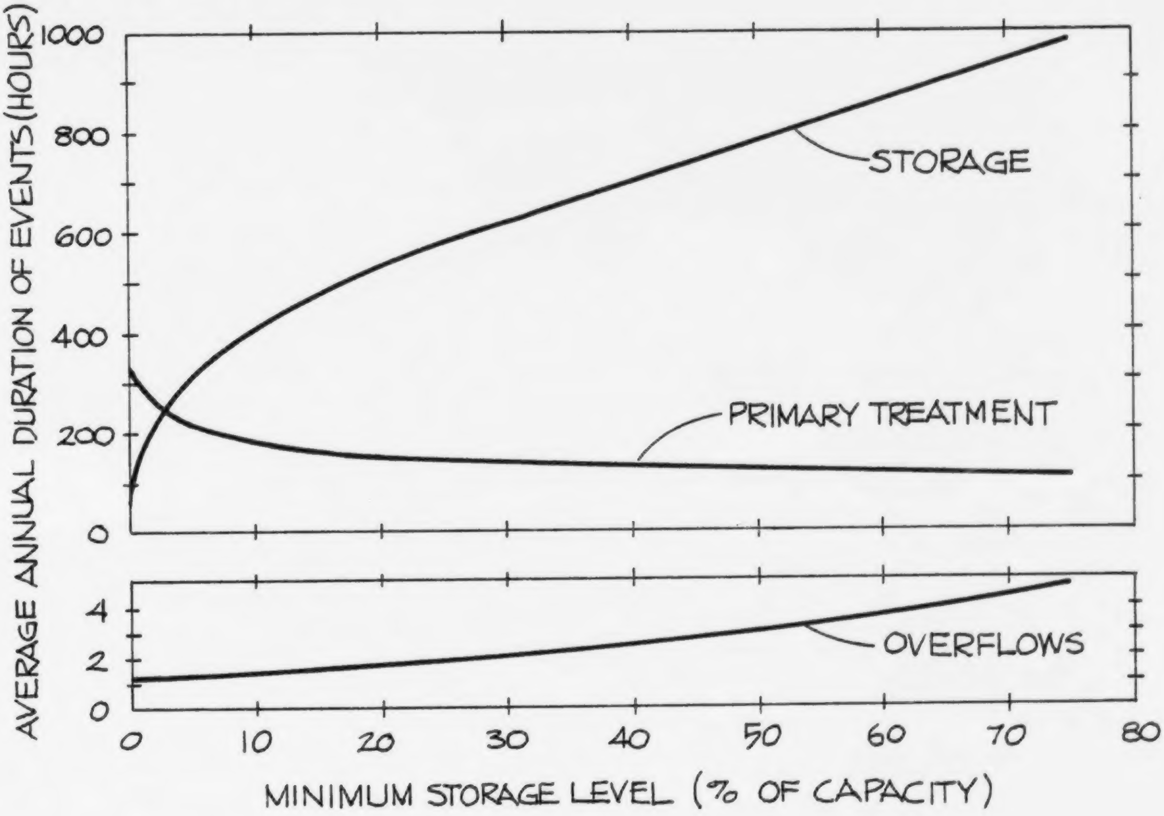


FIGURE VII-6

AVERAGE ANNUAL NUMBER OF HOURS OF STORAGE, TREATMENT AND OVERFLOWS VERSUS MINIMUM STORAGE LEVEL FOR ALTERNATIVE STRATEGY III BASIC OPERATING STRATEGY WITH MINIMUM STORAGE CONSTRAINTS the average duration of each storage event ranges from 2.7 hours to 18 hours as minimum storage levels vary from 3 to 75 percent of the storage capacity. Under the Basic Operating Strategy, the average storage event lasted 3.7 hours. The increased storage utilization is also reflected in the increase in maximum storage level and storage season as shown in Table VII-3.

A comparison of these results with those for Strategy II (Seasonal Constraints) shows that Strategy III is far more effective in all categories except for the maximum treatment season. This is understandable in view of the fact that the wet weather plant was assumed to be operational at all times in assessing the minimum storage level strategies. However, the previous results indicated two facts that may be significant with respect to further improvements on system performance: 1) operating the wet weather facilities for only certain months can improve system performance, and 2) the risk of overflows varies from month to month. These facts indicate the potential exists for further improvement in system performance by not operating the wet weather treatment plant during certain months, and by varying the minimum storage level from month to month based on overflow risk.

ALTERNATIVE IV - BASIC OPERATING STRATEGY WITH SEASONAL AND MINIMUM STORAGE CONSTRAINTS

Introduction

In view of the improvements over the Basic Operating Strategy made by Strategies II and III, Strategy IV was developed as a combination of the two. This strategy assumes the wet weather treatment plant would be shut down for those months in which the risk of overflows is low. In addition, separate minimum storage levels that must be reached before activation of the wet weather pumps and treatment plant would be established for each month in which the wet weather plant is in operation.

Development of Alternative Strategies

Because of the large number of possible combinations of monthly minimum storage levels and wet weather treatment seasons, it was not possible to test all feasible strategies. To develop a limited number of meaningful alternative strategies, some preliminary analyses were performed.

The preliminary analyses consisted of determining the number of overflows that could be expected in each month as a function of the minimum storage level. Given this information and the fact that the overflows in each month may be treated as independent from one another, it was possible to formulate detailed strategies that could be expected to produce known numbers of overflows using the model TREAT. Figure VII-7 presents the results of these preliminary TREAT runs. For every month, each point indicates that a TREAT run was made with the corresponding minimum storage level for that month. The number associated with each point shown on the figure is the number of overflows that were predicted for that month over the entire 70 year historical rainfall record. With this information, three strategies were developed; they are illustrated in Figures VII-8, VII-9 and VII-10 and described below.

Strategy	Name	Description
IV-A	Minimum Overflows	0.86 overflows/year - minimum number of overflows as in Basic Operating Strategy - 10 month plant operation
IV-B	One Overflow/year	1.00 overflow/year - meets overflow objective of 1/year - 8 month plant operation
IV-C	Seven Month Season	1.14 overflows/year - 7 month plant operation

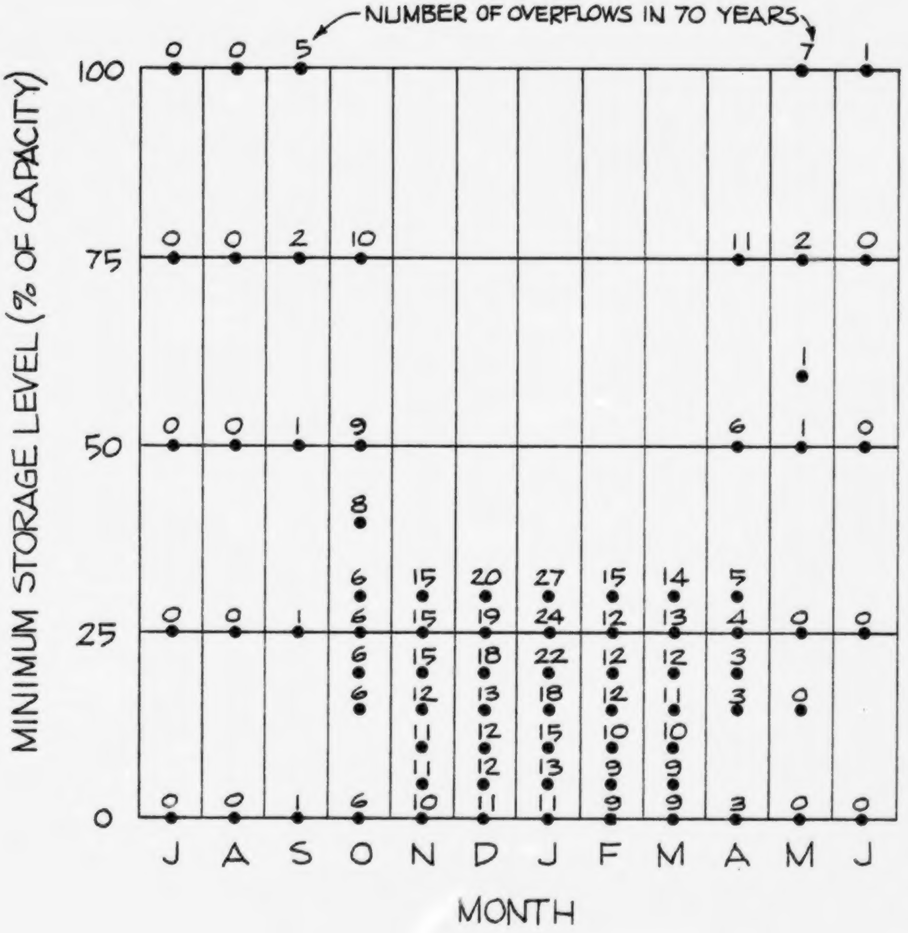


FIGURE VII-7

NUMBER OF OVERFLOWS VERSUS MONTH AND MINIMUM STORAGE LEVEL FOR ALTERNATIVE STRATEGY IV
BASIC OPERATING STRATEGY WITH SEASONAL AND MINIMUM STORAGE CONSTRAINTS

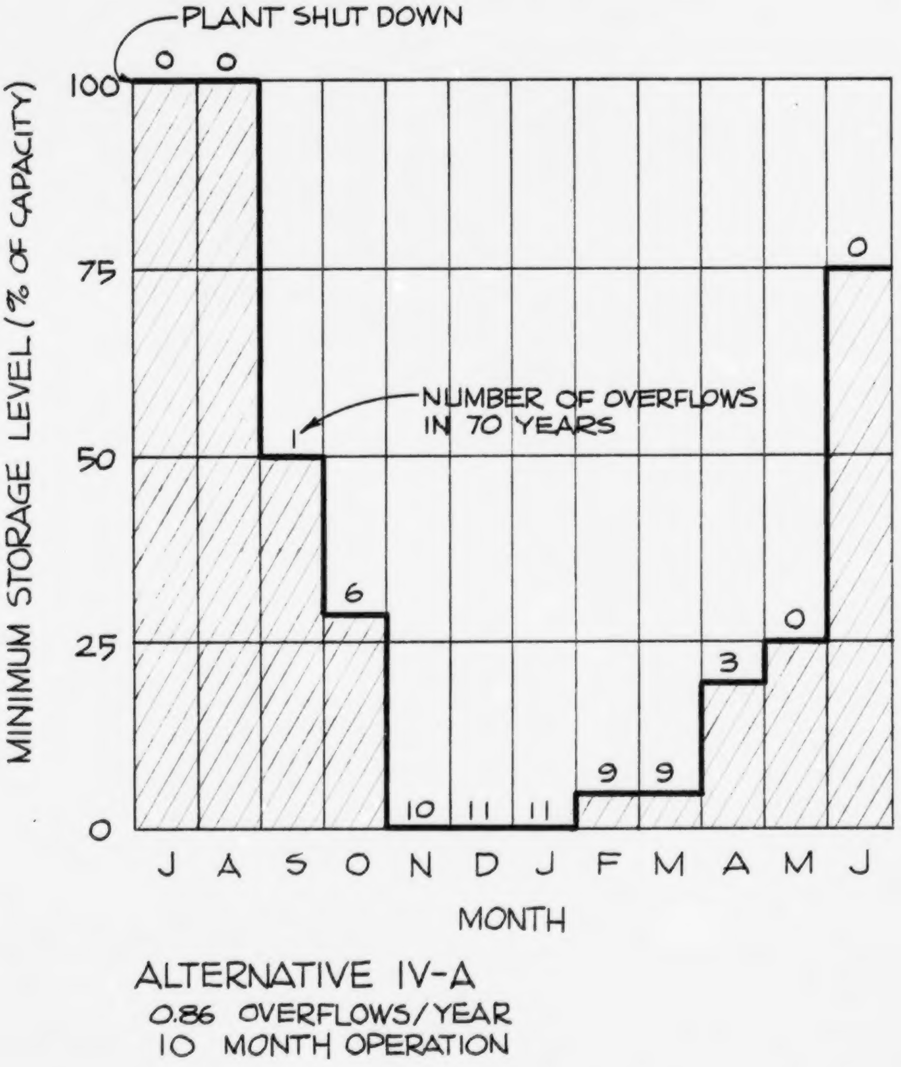


FIGURE VII-8

MONTHLY MINIMUM STORAGE LEVELS AND NUMBER OF OVERFLOWS FOR ALTERNATIVE STRATEGY IV-A
BASIC OPERATING STRATEGY WITH SEASONAL AND MINIMUM STORAGE CONSTRAINTS

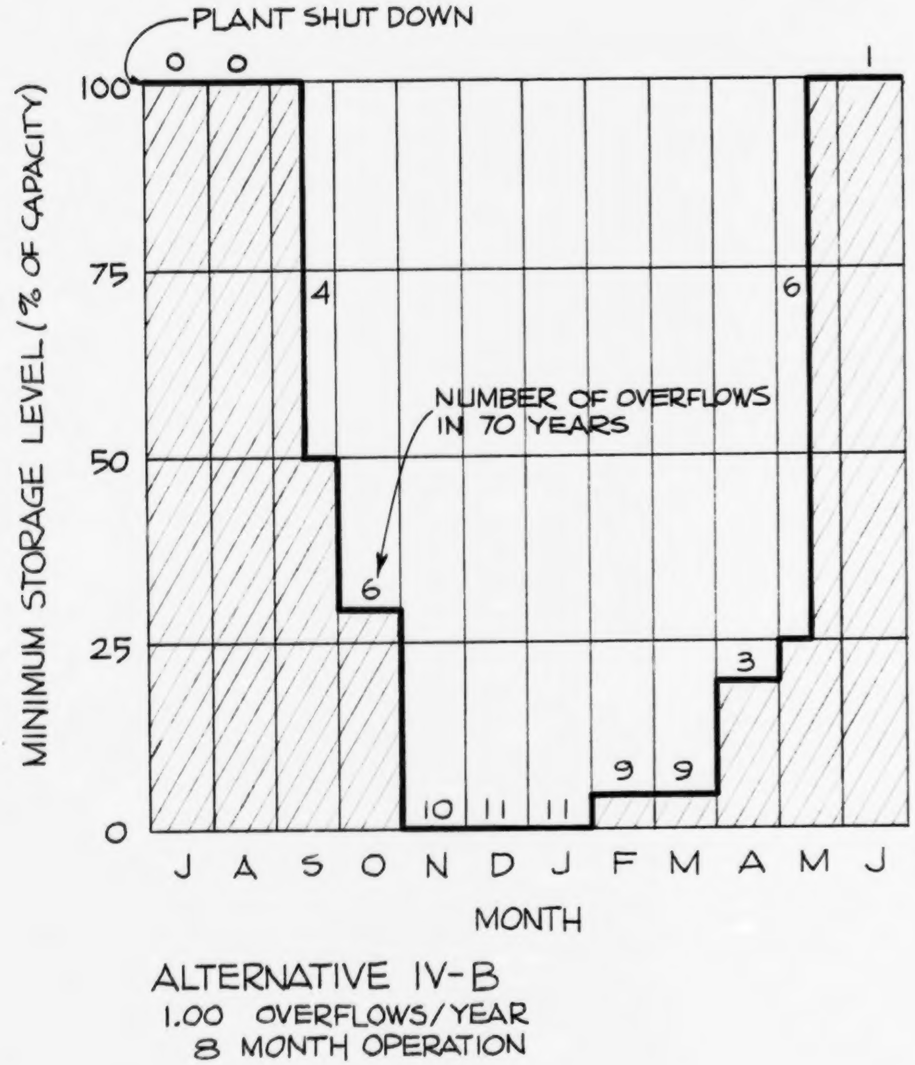
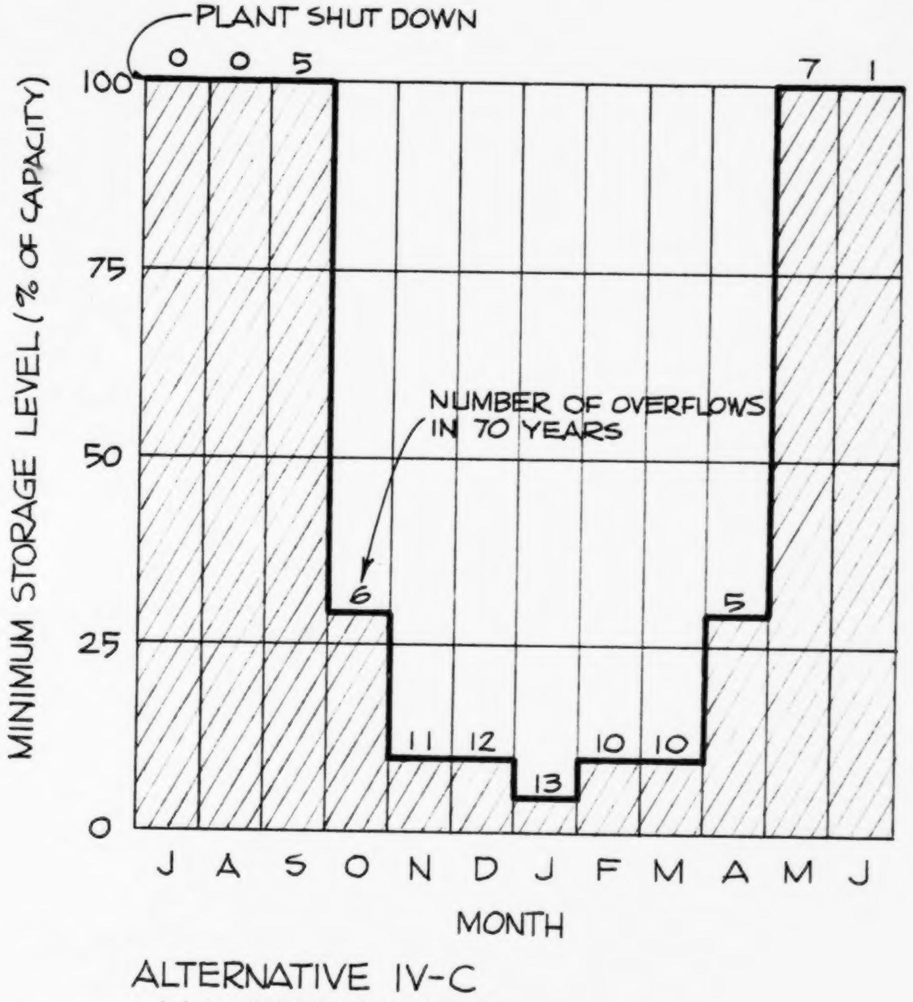


FIGURE VII-9

MONTHLY MINIMUM STORAGE LEVELS AND NUMBER OF OVERFLOWS FOR
ALTERNATIVE STRATEGY IV-B
BASIC OPERATING STRATEGY WITH SEASONAL AND MINIMUM STORAGE CONSTRAINTS



ALTERNATIVE IV-C
1.14 OVERFLOWS/YEAR
7 MONTH OPERATION

FIGURE VII-10

MONTHLY MINIMUM STORAGE LEVELS AND NUMBER OF OVERFLOWS FOR ALTERNATIVE STRATEGY IV-C
BASIC OPERATING STRATEGY WITH SEASONAL AND MINIMUM STORAGE CONSTRAINTS

In Figures VII-8 through VII-10, the heavy solid line represents the minimum storage levels for each month. The accompanying numbers are the expected number of overflows for each month for the 70 years of record. Strategy IV-A was developed from Figure VII-7 by finding the maximum storage level for each month that resulted in the same number of overflows as the Basic Operating Strategy, i.e., the minimum possible number of overflows. This strategy calls for the wet weather treatment plant to be operational in every month except July and August; these are the only months in which no overflows are predicted to occur. Strategy IV-B meets the overflow objective of one per year by operating the wet weather treatment plant between mid-September and mid-May for eight months of operation. Strategy IV-C calls for the wet weather plant to be operated from October through April (seven month operation) and raises the minimum storage levels in other months to achieve 80 overflows during the 70 years of record used in the analysis. Although other strategies could have been developed to achieve the recommended overflow objective of one per year, it was concluded that they would not be significantly different in their operational characteristics and, hence, would not impact the results of the cost effectiveness analysis. Therefore, the evaluations were limited to the three alternatives described above.

Simulation Results

Table VII-4 presents the average results for the three alternative strategies and the Basic Operating Strategy.

Strategy IV-A

This strategy results in the most improvement in performance over the Basic Operating Strategy that is possible without increasing the number of overflows. The performance is superior in every category save the amount and duration of untreated flows; these indicators increase modestly. The key

TABLE VII-4

SUMMARY OF RESULTS FOR ALTERNATIVE STRATEGY IV
BASIC OPERATING STRATEGY WITH SEASONAL AND MINIMUM STORAGE CONSTRAINTS
(MEAN ANNUAL VALUES)

PERFORMANCE INDICATOR	ALTERNATIVE STRATEGY			
	I	IV-A	IV-B	IV-C
Number of Events				
Use of Primary Treatment	106	75	75	47
Use of Storage	23	57	57	82
Untreated Overflows	0.86	0.86	1.0	1.1
Annual Amount of Events				
Secondary Treatment (MG)	7311	7435	7435	7549
Primary Treatment (MG)	1196	1070	1070	954
Untreated Overflows (MG)	8.9	9.8	11.3	12.7
Maximum Storage Level (MG)	24.0	24.4	24.4	24.8
Duration of Events				
Treatment Season (Days)*	365	303	242	212
Storage Season (Days)*	300	352	352	352
Primary Treatment Events (Hrs)	328	254	254	182
Storage Events (Hrs)	86	270	270	439
Untreated Overflows (Hrs)	1.2	1.3	1.8	2.0
Reduction in Pollutant Load (%)	-	3.5	3.4	6.8

*Maximum Value

improvements are a reduction in the number of times the wet weather treatment plant must be placed in operation, an improved level of treatment, more effective use of storage, a shortened wet weather treatment season, and a reduction in the pollutant load that would be discharged to San Francisco Bay. The extent of these improvements, however, is not as great as for many of the alternative strategies previously evaluated. This lack of improvement is because the strategy is constrained so the number of overflows will not increase as compared to the Basic Operating Strategy; this constraint is a costly one in terms of performance in other categories, since it forces the minimum storage levels to be zero, or very low for several months of the year and requires the wet weather treatment plant to be operational ten months of the year.

Strategy IV-B

This strategy differs from Strategy IV-A in that is restricts plant operations to eight months instead of ten. The only notable effect other than the reduced length of the wet weather season is a slight increase in overflow numbers, amount and duration. The virtually identical values in all other performance categories demonstrates that the reduction in plant operations of two months is surely a cost-effective measure.

Strategy IV-C

This strategy further reduces the wet weather treatment season to seven months and raises the minimum storage levels to ten percent of storage capacity in most of the wet weather months. The effects of these changes are a minor increase in the frequency of untreated overflows (to 1.14/year) and an accompanying increase in amount and duration of overflows. Other operating improvements, however, make this strategy appear very desirable on balance. The level of treatment improves significantly as indicated by the sizable reduction in pollutant load entering the Bay. The number of times the wet weather treatment plant must be started drops drastically and the use of storage improves substantially in all categories. Most of the improved performance of this strategy can be atrributed to the increase in minimum storage levels to the 5-10 percent level or higher in all months.

The results of Strategy IV, and the knowledge gained from the analyses of Strategies II and III indicate that further system improvements are possible by further shortening the treatment season and by raising the minimum storage levels. However, these changes would result in violation of the one overflow per year constraint. Without such a constraint, a trade-off would be required between further system improvements and increased overflows.

ALTERNATIVE V - BASIC OPERATING STRATEGY WITH RAINFALL PREDICTION AND SEASONAL AND MINIMUM STORAGE CONSTRAINTS

Introduction

The subject of rainfall prediction and its relation to real time automatic control was introduced in Chapter VI. The potential advantages of rainfall prediction can be seen by considering five categories of storms that might occur and comparing the differences in system performance between predictive and reactive strategies.

1. *Small storms that do not exceed the minimum storage level for activation of the wet weather pumps (as defined in Alternative Strategy III).* In this case predictive strategies would offer no advantages. Wet-weather facilities would not be needed in either a reactive or predictive mode.

2. *Storms that exceed the minimum storage level, but would not overflow if the wet weather pumps were left off.* The prediction that an overflow would not occur could avoid the unnecessary start-up of the wet weather plant and result in secondary treatment of all stormwater.

3. *Storms that exceed the minimum storage level and would overflow unless the wet weather pumps are turned on after the minimum storage level is reached.* Rainfall prediction could delay the activation of the wet-weather pumps, increasing secondary treatment amounts, yet still avoiding overflows.

4. *Storms that exceed the minimum storage level and would overflow unless the wet weather pumps are turned on before the minimum storage level is reached.* Rainfall prediction could lead to the early activation of the wet weather pumps and avoidance of overflows, but with reductions in the amount of flow receiving secondary treatment.

5. *Storms that exceed the minimum storage level and would overflow even if the wet weather pumps were turned on at the start of the storm.* Rainfall prediction could lead to the early activation of the wet weather pumps and reductions in the volume of overflow, which would also reduce the amount of flow receiving secondary treatment.

To assess the potential of rainfall prediction in system control, the following approach was taken:

1. A control algorithm was developed that incorporated both rainfall prediction and reactive rules, and was included in the program TREAT.

2. A file of perfect rainfall predictions was prepared from the recorded historical 70-year rainfall data file. The rainfall prediction model RAFORT was not used in this step; the "predictions" are actual rainfall amounts that occurred for a given number of hours following each simulated hour. This data set is, therefore, a "perfect" prediction, and the performance of the system using these "perfect" predictions defines an upper limit on the improvements that would be possible using actual rainfall prediction. Perfect prediction files were prepared for forecasts of 2, 3, 4 and 5 hours into the future to test the effect of forecast times.

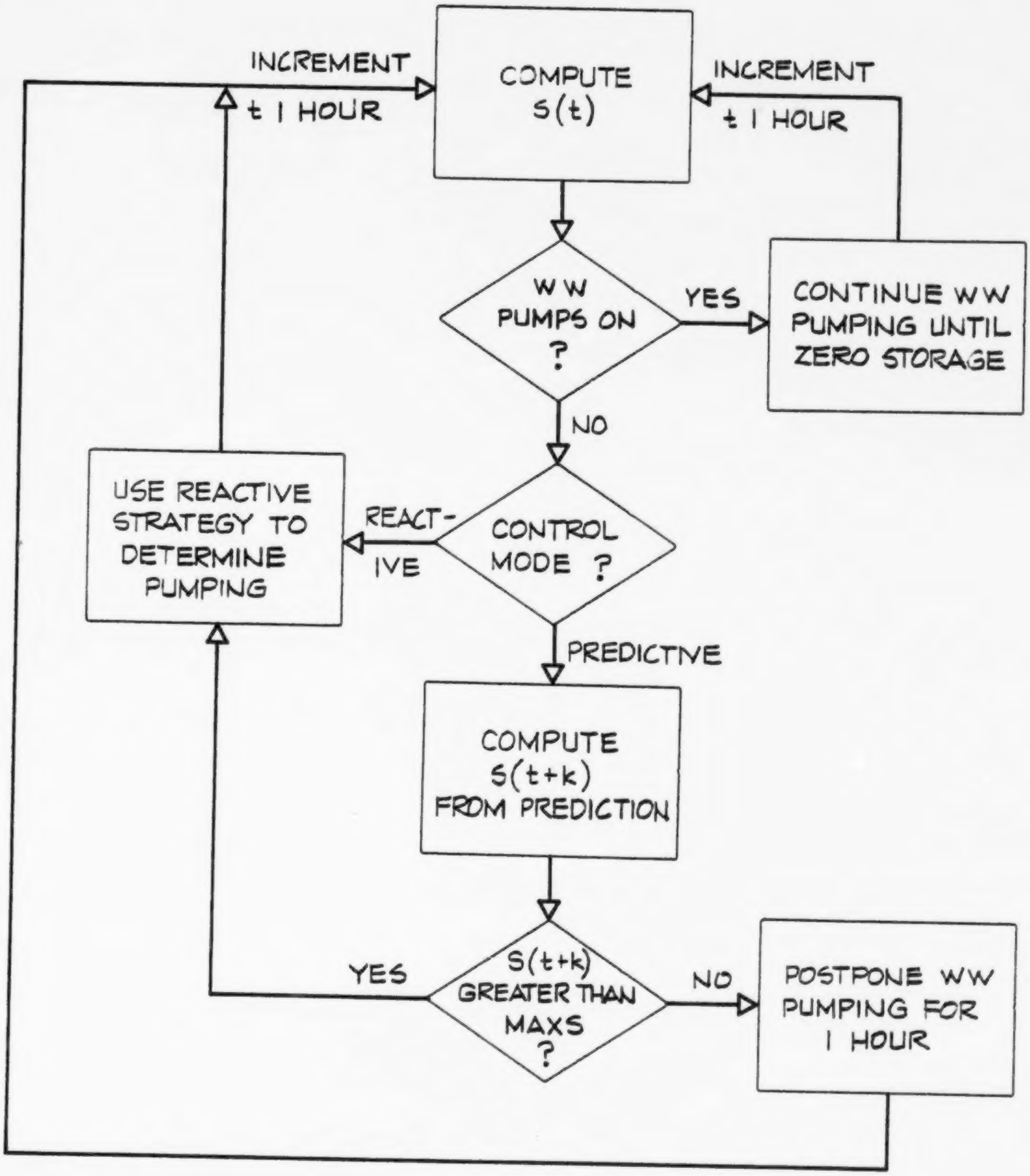
3. A file of actual predictions was prepared using the RAFORT model with actual historical rainfall at both remote and local gages. Due to limited data availability and as discussed in Chapter VI, only 28 years of rainfall predictions could be made. The forecast times for predictions are computed by RAFORT and are variable from hour to hour within the storm.

4. Several alternative strategies incorporating rainfall prediction were developed and tested using the modified TREAT model and the "perfect" and actual rainfall prediction files to assess the improvements in performance that could be expected through the use of rainfall prediction.

Development of Alternative Strategies

The algorithm shown in Figure VII-11 was developed to test the effects of rainfall prediction on the NSOC Project performance. The algorithm says that a chosen reactive strategy (any reactive strategy, such as Alternative IV) will be followed unless the rainfall prediction and corresponding performance indicates it should be overridden. The performance prediction could result in control decisions for the wet weather pumps to be turned on sooner or later than the reactive strategy would dictate. The strategy calls for a prediction to be made at each hour of the amount of stormwater in storage in k hours if the dry-weather pumps are used for one hour and the wet weather pumps are turned on for the remaining k-1 hours. This predicted storage level is then compared to a value of storage called MAXS. MAXS is a limiting storage level that must be determined during the evaluation of alternative strategies. It can take any value from 0 to the storage capacity of the system. If the predicted storage level exceeds MAXS, control reverts to the reactive strategy and wet weather plant activation is controlled by the designated minimum storage level. If the predicted storage level is less than MAXS, wet weather pumping would be delayed for 1 hour and the reactive strategy would be overridden.

Higher values of MAXS delay the start of pumping longer, but are "riskier" than lower values in terms of allowing overflows. However, they may result in better system performance by increasing the use of secondary treatment and reducing the use of primary treatment. The minimum storage levels for the reactive strategy dictate how soon the wet weather pumping would start when the prediction shows MAXS will be exceeded after k hours. Lower minimum storage levels will, therefore, minimize the risk of overflows but also reduce the level of treatment. There is a combination of a MAXS value (or values by month) and a reactive strategy that would produce the best performance, based on the particular system's response and objectives.


INCREMENT
t 1 HOUR
COMPUTE S(t)
INCREMENT
t 1 HOUR
WW PUMPS ON ?
YES
CONTINUE WW PUMPING UNTIL ZERO STORAGE
NO
USE REACTIVE STRATEGY TO DETERMINE PUMPING
REACT-IVE
CONTROL MODE ?
PREDICTIVE
COMPUTE S(t+k) FROM PREDICTION
YES
S(t+k) GREATER THAN MAXS ?
NO
POSTPONE WW PUMPING FOR 1 HOUR

WW = WET WEATHER

MAXS = USER INPUT STORAGE PARAMETER ($0 \leq MAXS \leq$ STORAGE CAPACITY)

$S(t)$ = AMOUNT OF FLOW IN STORAGE AT TIME t

k = LENGTH OF PREDICTION IN HOURS, OR LAG

$S(t+k)$ = AMOUNT OF FLOW EXPECTED IN STORAGE AT TIME $(t+k)$ IF DRY WEATHER PUMPS ARE USED FOR FIRST HOUR AND WET WEATHER PUMPS ARE USED THEREAFTER

FIGURE VII-11

CONTROL ALGORITHM INCORPORATING RAINFALL PREDICTION
ALTERNATIVE STRATEGY V
BASIC OPERATING STRATEGY WITH RAINFALL PREDICTION AND SEASONAL AND MINIMUM STORAGE CONSTRAINTS

The goal in this study was not to find such an optimal solution. Instead, four sets of alternative strategies were formulated to investigate the effect of varying the following factors:

1. Forecast time or lag (k)
2. Maximum or limiting control storage (MAXS)
3. Reactive strategy
4. Actual vs perfect prediction

The strategies in all four groups are defined in Tables VII-5 through VII-8. The analysis of the results are presented by strategy set in the following sections.

Strategy Set 1 - Variations In Forecast Times With Perfect Prediction

Table VII-5 shows these results. Control decisions for all runs were based on perfect predictions for k hours into the future, with the forecast time (k) varying from 2 to 5 hours. The reactive strategy employed was Alternative IV-C, with monthly minimum storage constraints. The first column gives the results from the reactive run with no rainfall prediction for comparison.

All four "perfect" predictive runs reduced the average number of primary treatment events to 38 times per year. Overflows increased slightly for the 2 hour lag run because the delay of wet-weather pumping resulted in some additional overflows. Since the same minimum storage levels were used in the reactive Alternative IV-C and the predictive runs, the pumps could not start sooner than they did for the reactive run, and so no overflows were prevented from the larger storms.

TABLE VII-5

SUMMARY OF RESULTS FOR ALTERNATIVE STRATEGY V
BASIC OPERATING STRATEGY WITH RAINFALL PREDICTION AND SEASONAL AND MINIMUM STORAGE CONSTRAINTS
(MEAN ANNUAL VALUES)
STRATEGY SET 1
VARIATIONS IN FORECAST TIMES WITH PERFECT PREDICTION

STRATEGY REACTIVE STRATEGY CONTROL MODE LAG (HRS) MAXS (FRACTION CAPACITY)	IV-C - REACTIVE - -	V-A1 IV-C PREDICTIVE 2 0.2	V-A2 IV-C PREDICTIVE 3 0.2	V-A3 IV-C PREDICTIVE 4 0.2	V-A4 IV-C PREDICTIVE 5 0.2
Performance Indicator					
Number of Events					
Use of Primary Treatment	47	38	38	38	38
Use of Storage	82	73	73	74	74
Untreated Overflows	1.14	1.21	1.16	1.17	1.16
Annual Amount of Events					
Secondary Treatment (MG)	7549	7597	7598	7598	7598
Primary Treatment (MG)	954	905	905	905	905
Untreated Overflows (MG)	12.7	13.1	12.7	12.6	12.6
Maximum Storage Level (MG)	24.8	25.0	24.9	24.8	24.8
Duration of Events					
Treatment Season (Days)*	212	212	212	212	212
Storage Season (Days)*	261	261	261	261	261
Primary Treatment Events (Hrs)	182	164	163	163	163
Storage Events (Hrs)	439	521	521	521	521
Untreated Overflows (Hrs)	2.0	2.1	2.1	2.1	2.1
Reduction in Pollutant Load (%)	6.8	7.7	7.7	7.7	7.7

---

*Maximum Value

Extending the lag from 2 to 3 hours succeeded in preventing almost all of the added overflows. Further lengthening of the lag had no significant effect on any performance indicators. Therefore, it may be concluded that a lag or forecast time of 3 hours is the longest that is useful for prediction, at least as far as improvements resulting from delayed wet weather pumping are concerned. Consequently, a forecast time of 3 hours was used in subsequent evaluations using "perfect" rainfall predictions.

Strategy Set 2 - MAXS Variations With Perfect Prediction

Increasing MAXS results in longer delays before wet weather pumping would be started. As expected, Table VII-6 shows that we can reduce the number of primary treatment events greatly and reduce the pollutant load by increasing MAXS, but we would also increase the number of overflows. The effect is the same as discussed above; smaller storms get better treatment, but larger storms result in more overflows. The results indicate a value of MAXS in the range of 0.2 to 0.4 yields the best results.

Strategy Set 3 - Reactive Strategy Variation With Perfect Prediction

The only way to reduce the number of overflows resulting from the larger storms is to allow the wet weather pumps to come on sooner in cases where the rainfall prediction indicates a high likelihood of overflow (i.e., predicted storage greater than MAXS). In the algorithm employed, control reverts to the reactive strategy in this case. Therefore, using a reactive strategy which has lower minimum storage levels for activating wet weather treatment facilities should improve the results.

TABLE VII-6

SUMMARY OF RESULTS FOR ALTERNATIVE STRATEGY V
BASIC OPERATING STRATEGY WITH RAINFALL PREDICTION AND SEASONAL AND MINIMUM STORAGE CONSTRAINTS
(MEAN ANNUAL VALUES)
STRATEGY SET 2
MAXS VARIATIONS WITH PERFECT PREDICTION

STRATEGY	IV-C	V-A2	V-B	V-C
REACTIVE STRATEGY	-	IV-C	IV-C	IV-C
CONTROL MODE	REACTIVE	PREDICTIVE	PREDICTIVE	PREDICTIVE
LAG (HRS)	-	3	3	3
MAXS (FRACTION CAPACITY)	-	0.2	0.4	1.0
Performance Indicator				
Number of Events				
Use of Primary Treatment	47	38	27	14
Use of Storage	82	73	63	48
Untreated Overflows	1.14	1.16	1.29	6.06
Annual Amount of Events				
Secondary Treatment (MG)	7549	7598	7685	7873
Primary Treatment (MG)	954	905	817	611
Untreated Overflows (MG)	12.7	12.7	13.9	32.3
Maximum Storage Level (MG)	24.8	24.9	25.4	27.4
Duration of Events				
Treatment Season (Days)*	212	212	212	212
Storage Season (Days)*	261	261	261	261
Primary Treatment Events (Hrs)	182	163	139	99
Storage Events (Hrs)	439	521	675	>1000
Untreated Overflows (Hrs)	2.0	2.1	2.2	5.9
Reduction in Pollutant Load (%)	6.8	7.7	9.1	11.5

*Maximum Value

Table VII-7 presents the results of using Reactive Strategies I and II-C instead of IV-C in the reactive control algorithm. Strategy I sets the minimum storage level at 0 and Strategy II-C sets it at 0 for the seven wet months and at the system capacity during dry months (i.e., shut down). In both cases the number of primary treatment events and the pollutant reduction were almost the same as Strategy IV-C with perfect prediction, but the number of overflows was substantially lower. Without losing the benefits of delayed start-up for smaller storms, some overflows were prevented. As shown by Strategy V-F, this reduction in the number of overflows allows an increase in MAXS to be made to further reduce the number of primary treatment events and the pollutant load, while still providing adequate overflow control. This strategy (V-F) demonstrates the potential improvement in system performance that could be achieved if we could predict rainfall perfectly. The following set of strategies were evaluated to find out how much improvement we could expect by using the existing rainfall prediction model RAFORT.

Strategy Set 4 - Actual Versus Perfect Prediction

All the preceeding predictive runs used perfect rainfall prediction. These perfect predictions were taken from the historical 70-year record and the results represent the best performance that can be expected for the given strategy and algorithm employed.

To compare the RAFORT predictions to the perfect predictions, a 28-year set of RAFORT predictions was constructed and used as input to TREAT as Strategy V-G* The reactive strategy was IV-C*(same as IV-C, but overthe 28-year period rather than 70 years). The Reactive Run IV-C was also re-run for 28 years and is referred to as IV-C* The results are shown in Table VII-8.

TABLE VII-7

SUMMARY OF RESULTS FOR ALTERNATIVE STRATEGY V
BASIC OPERATING STRATEGY WITH RAINFALL PREDICTION AND SEASONAL AND MINIMUM STORAGE CONSTRAINTS
(MEAN ANNUAL VALUES)

STRATEGY SET 3

REACTIVE STRATEGY VARIATIONS WITH PERFECT PREDICTION

STRATEGY	IV-C	V-A2	V-D	V-E	V-F
REACTIVE STRATEGY	-	IV-C	I	II-C	I
CONTROL MODE	REACTIVE	PREDICTIVE	PREDICTIVE	PREDICTIVE	PREDICTIVE
LAG (HRS)	-	3	3	3	3
MAXS (FRACTION CAPACITY)	-	0.2	0.2	0.2	0.4
Performance Indicator					
Number of Events					
Use of Primary Treatment	47	38	38	39	25
Use of Storage	82	73	75	75	63
Untreated Overflows	1.14	1.16	0.90	1.07	1.04
Annual Amount of Events					
Secondary Treatment (MG)	7549	7598	7589	7593	7687
Primary Treatment (MG)	954	905	918	911	818
Untreated Overflows (MG)	12.7	12.7	9.2	11.3	10.8
Maximum Storage Level (MG)	24.8	24.9	24.1	24.4	25.0
Duration of Events					
Treatment of Season (Days)*	212	212	365	212	365
Storage Season (Days)*	261	261	261	261	261
Primary Treatment Events (Hrs)	182	163	168	166	139
Storage Events (Hrs)	439	521	506	516	678
Untreated Overflows (Hrs)	2.0	2.1	1.3	1.9	1.5
Reduction in Pollutant Load	6.8	7.7	7.5	7.6	9.1

*Maximum Value

Table VII-8 shows that the 28-year record contains 0.1 fewer overflows per year than the 70-year record, but is quite comparable for other indicators. The perfect prediction Strategy V-A2*also compares closely to its 70-year record comparison Run V-A2, except that there are 0.09 fewer overflows per year. These results demonstrate comparable performance for the two periods with the exception of overflows.

The results from the RAFORT prediction (Run V-G*) compared to the perfect predictions (Run V-A2*) indicate that the small storms are detected and use of secondary treatment and the pollutant load reductions are virtually the same. However, the uncertainty in the predictions has caused an increase in the number of overflows because of delayed start-up of the wet weather pumps (1.43 vs 1.07 per year). The larger storms are probably unaffected by the accuracy of predictions, since those that overflowed under Strategy IV-C*also overflowed under Strategy V-A2*. The increase in overflows in Strategy V-G* apparently results from inaccuracies in predicting moderate to large storms.

Utilizing a reactive strategy having lower minimum storage levels might lower the number of overflows by about 0.1 per year. Therefore, it appears that a reduction in overflows to much less than 1.4 per year would require a reduction in MAXS, and a corresponding decrease in the other performance indicators would result. It would appear from these results that the performance of the NSOC system using the RAFORT predictions would be perhaps only slightly better than the performance using a good reactive strategy with no prediction. Improvements in the control algorithm and in the accuracy of the predictions could result in additional performance improvements. Also, if the number of allowable overflows is allowed to be substantially greater than one per year, the predictions may have a more significant beneficial effect on the system's performance, although this has not been tested or demonstrated by simulations.

TABLE VII-8

SUMMARY OF RESULTS FOR ALTERNATIVE STRATEGY V
BASIC OPERATING STRATEGY WITH RAINFALL PREDICTION AND
SEASONAL AND MINIMUM STORAGE CONSTRAINTS
(MEAN ANNUAL VALUES)
STRATEGY SET 4
ACTUAL VERSUS PERFECT PREDICTION
*28 YEAR SIMULATION

STRATEGY	IV-C	IV-C*	V-A2*	V-G*
REACTIVE STRATEGY	-	-	IV-C*	IV-C*
CONTROL MODE	REACTIVE	REACTIVE	PREDICTIVE	RAFORT PREDICTION
LAG (HRS)	-	-	3	VARIABLE
MAXS (FRACTION CAPACITY)	-	-	0.2	0.2
Performance Indicator				
Number of Events				
Use of Primary Treatment	47	48	38	38
Use of Storage	82	85	76	75
Untreated Overflows	1.14	1.04	1.07	1.43
Annual Amount of Events				
Secondary Treatment (MG)	7549	7551	7606	7603
Primary Treatment (MG)	954	937	882	884
Untreated Overflows (MG)	12.7	14.4	14.6	16.1
Maximum Storage Level (MG)	24.8	24.1	24.2	24.6
Duration of Events				
Treatment Season (Days)*	212	212	212	212
Storage Season (Days)*	261	270	270	260
Primary Treatment Events (Hrs)	182	183	160	161
Storage Events (Hrs)	439	437	532	517
Untreated Overflows (Hrs)	2.0	1.9	1.9	2.2
Reduction in Pollutant Load (%)	6.8	6.8	8.0	7.9

*Maximum Value

COST OF ALTERNATIVES

General

The following cost estimates for the alternative North Shore RTAC Systems show the estimated budget requirements for capital and operational costs associated with proposed equipment purchase, construction, installation, operation, and maintenance. Estimated prices are as of January 1, 1979.

The objective of the cost comparison is to show estimated, annual cost differences between the five control strategies and between manual and automatic control implementation of the Basic Operating Strategy. Although several different strategies were analyzed under Alternative Strategies II through V, only strategies producing less than 1.1 overflows per year were selected in order to provide a common basis of comparison among the alternatives.

Cost comparisons were made on the basis of equivalent uniform annual costs or simply annual costs. The capital costs were evaluated on the basis of twenty year life, using an annual interest rate of 6 7/8 percent. Operational costs were evaluated on the basis that estimated annual costs are average costs per year and, therefore, are a uniform annual series.

It is important to note that the capital costs reflect financing of the entire project, whereas in actuality, a considerable portion of the capital costs would more than likely be eligible for funding by federal and local agencies. In the same light, operational costs play a significant role because the City is responsible for 100 percent of the operation and maintenance expenditures.

Capital Costs

The capital costs associated with implementing the RTAC system for the North Shore are summarized in Table VII-9. It should be noted that the costs shown are for equipment and construction only and, with one exception, do not include any engineering or contingency costs associated with the engineering design of the system. The exception is Strategy V. In this case we have included the engineering costs associated with software development for models to be used in the predictive control system. A description of each item follows.

1. Inflatable Dams - Furnish and install four inflatable dams, one at each of the four priority overflow points. Equipment for each will include the dam, blower, pipes, controls and materials necessary to construct a small vault below grade to house the blower and controls. Installation includes construction of the vault.

2. Flushing Gates - Furnish two sluice gates with hydraulic or motor operators, and install them in existing (other NSOC Contracts) gate structures with controls.

3. Level Monitors - Furnish and install nine level monitors. Each monitor will consist of two level transmitters that will be compared for reasonability. This will provide redundancy, as sewer level or volume is the main control parameter.

4. Flow Monitors - Furnish and install four level sensing devices installed behind the existing (other NSOC Contracts) weirs, located at each overflow point. The system will calculate flow from water level.

TABLE VII-9

CAPITAL COST ESTIMATE FOR ALTERNATIVE CONTROL STRATEGIES FOR THE NSOC PROJECT

Item	Manual Control	Strategy I	Strategy II	Strategy III	Strategy IV	Strategy V
1. Inflatable Dams	$ 90,000	$ 90,000	$ 90,000	$ 90,000	$ 90,000	$ 90,000
2. Flushing Gates	58,500	58,500	58,500	58,500	58,500	58,500
3. Level Monitors	19,500	19,500	19,500	19,500	19,500	19,500
4. Flow Monitors	5,500	5,500	5,500	5,500	5,500	5,500
5. Wiring	27,500	27,500	27,500	27,500	27,500	27,500
6. Data Logging	54,500	54,500	54,500	54,500	54,500	54,500
7. Control	8,500	10,000	11,500	11,500	11,500	53,500
8. Software	-0-	-0-	-0-	-0-	-0-	150,000
TOTAL CAPITAL COST	264,000	265,500	277,000	277,000	277,000	459,000
ANNUAL COST	24,700	24,800	25,900	25,900	25,900	42,900

$$P \left[\frac{i(1+i)^{n}}{(1+i)^{n}-1} \right]$$

$i = .06875$
$n = 20$

crf = .09348

5. Wiring - Furnish and install the electrical wiring required to connect flow, level, and gate status monitors and controls to the control center. Wiring to be installed in existing (other NSOC Contracts) conduits.

6. Data Logging - Furnish and install all hardware including microprocessor based central processing unit, memory, magnetic device for loading system, magnetic device for historical storage, color cathode ray tube (CRT) for displays, functional key pad for operator interface, signal input hardware for accepting process inputs, medium speed printer for logs and reports, and a data link to the future central control facility. Furnish and test all operating system and applications software related to data logging.

7. Control

   a) Automatic Reactive Control* - Furnish and install the necessary output signal hardware, emergency manual back-up panel, and the additional application software to perform the reactive control strategies described in Chapter IV.

   b) Automatic Reactive-Predictive Control** - Furnish and install the above reactive control hardware and software and the additional hardware (processing capability and memory) and the additional software (programming of rainfall prediction and runoff/transport models running in real time) to implement the integrated reactive-predictive control strategy described in Chapter IV.

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* - Applies to Strategies I, II, III and IV
** - Applies to Strategy V

c) Manual Control*** - Furnish and install a full-scale manual control panel with switches, pushbuttons, indicators, controls, and an annunciator system for manual control of the North Shore system.

8. Software - Develop and calibrate software (mathematical models) that would be needed to implement a predictive strategy. The software would include models for rainfall prediction, runoff estimation, and system hydraulic performance.

Table VII-9 shows that capital costs for all strategies, except Strategy V, are virtually the same. Strategy V is more capital intensive as a result of the additional need for software development.

Operational Cost

Operation and maintenance costs associated with the RTAC system for the North Shore are very broad in nature and would have to include costs of secondary treatment vs. primary treatment; costs of dry weather pumping vs. wet weather pumping; and costs of overflowing vs. pumping and treating. Because this would require a very complex economic analysis that would be subject to error and is beyond the scope of this report, the following assumptions were made:

1. The primary object of this cost analysis is to show the *relative* differences in costs of implementing and operating each of the five proposed strategies.

2. The main objective of the RTAC system is to limit overflows and provide the best treatment possible for the pollution reduction. Therefore, any electrical,

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***Applies to manual control of Strategies I, II, III, and IV

Costs do not include hardware and software costs specific to Reactive-Predictive Control (Strategy V) although this control strategy could be manually controlled as well.

chemical, or miscellaneous treatment costs associated with providing secondary treatment vs. primary treatment vs. non-treated overflows can be considered an additional benefit.

3. Based on previous San Francisco Wastewater Treatment Division and Wastewater Pumping Division annual budget reports, the power costs are only 5 percent of the total O&M, while labor costs are 60-80 percent of the total. Therefore, labor costs are by far the significant factor and power costs were ignored.

4. The personnel required to operate the North Point Plant is significant to the RTAC system cost comparison, as different strategies require staffing the plant for different months of the year. Costs were estimated to show the increase or decrease in this cost element relative to manual control.

5. The North Shore Pump Station will require some staffing whether the RTAC system is implemented or not. The staffing shown in the RTAC cost comparison is for additional control facility operations personnel required for the RTAC system.

6. The North Shore sewer system will require work crews, trucks, and equipment whether the RTAC system is implemented or not. The staffing shown in the RTAC cost comparison is for additional maintenance personnel required for the RTAC system.

The remaining operational costs associated with the RTAC system for the North Shore are summarized in Table VII-10. A description of each item follows.

TABLE VII-10

O&M COST ESTIMATES FOR ALTERNATIVE
CONTROL STRATEGIES FOR THE NSOC PROJECT

Item	*Manual Control	Strategy I	Strategy II	Strategy III	Strategy IV	Strategy V
1. Treatment Season	365 days	365 days	212 days	365 days	212 days	212 days
2. NPWPCP Staffing	base = 0	-0-	⟨$334,200⟩	-0-	⟨$334,200⟩	⟨$334,200⟩
3. R.T.A.C. Staffing	$262,800	$131,400	131,400	$131,400	131,400	131,400
4. R.T.A.C. Maint.	35,600	35,700	35,800	35,800	35,800	37,900
O&M ANNUAL COST	298,400	167,100	⟨167,000⟩	167,200	⟨167,000⟩	⟨166,900⟩

* The manual control costs are shown for operating Strategy I manually by operator decision.

1. Treatment Season - The number of days per year that the North Point Water Pollution Control Plant (NPWPCP) must be in operation for the particular strategy.

2. NPWPCP Staffing - During the treatment season for each strategy, we have assumed seven additional persons per shift, around the clock. An average hourly salary of $13.00 was used. The figures shown represent the cost difference between the automatic control strategies and manual control. Hence, the costs are the incremental costs (or savings) associated with automatic control.

3. RTAC Staffing - We estimate the need of one person per shift for the automatic strategies and two persons per shift for the manual control, located at the control facility. An average hourly salary of $15.00 was used. Continuous staffing is assumed.

4. RTAC Maintenance - We estimate the need of one person working 2080 hours per year to perform specialized maintenance on the monitoring devices and control equipment. An average hourly salary of $15.00 was used. Experience shows that average annual expenditures for parts and specialized outside help can be approximated at 5 percent of the initial cost for such things as instrumentation and computer controls on a project of this magnitude. It is assumed that the structural and mechanical maintenance required on the gates and dams can be accomplished by the regular sewer maintenance crew. Although these additional facilities may increase the time and materials otherwise used, it is felt that this will be more than offset by the RTAC system's flushing capabilities, which will decrease the amount of time required for flushing by the sewer crew.

Table VII-10 shows manual control to have the highest operational costs with Strategies II, IV and V resulting in operational cost savings. It should be noted that these savings are the result of reduced staffing needs when the NPWPCP is not in operation. The practical problem remains of what to do with operations personnel during extended periods when the plant is not in operation. Presumably, they can be used for maintenance purposes or in other assignments in the City wastewater program.

Summary of Cost Estimates

Table VII-11 summarizes the total annual costs for the five alternative automatic control strategies as well as manual control. The following observations summarize the cost estimates presented in Table VII-11.

1. Manual control is more costly than automatic control because the annual cost of additional personnel required for manual control is greater than the annual capital cost of the digital control equipment needed to implement automatic control.

2. Strategies I and III have high costs because of the need to staff the treatment plant for twelve months of the year.

3. There is no difference in cost between Strategies II and IV since they only differ in the rule used to start the wet weather treatment facilities.

4. The additional cost of Strategy V over Strategies II and IV is the cost of additional computer hardware and software (including maintenance) required to implement an integrated reactive-predictive control strategy.

TABLE VII-11

TOTAL ANNUAL COSTS FOR ALTERNATIVE CONTROL STRATEGIES FOR THE NSOC PROJECT

Strategy	Annual Capital Cost	Annual O&M Cost	Total Annual Cost
Manual	$24,700	$298,400	$323,100
I	24,800	167,100	191,900
II	25,900	<167,000>	<141,100>
III	25,900	167,200	193,100
IV	25,900	<167,000>	<141,100>
V	42,900	<166,900>	<124,000>

5. Alternatives II and IV result in net cost reductions and are the least costly alternatives. This finding is sensitive to the assumption concerning use of operational personnel during periods the wet weather facilities are not in use.

COST-EFFECTIVENESS

From the 13 performance indicators used in the evaluation of alternatives, we selected 7 for inclusion in the cost-effectiveness analysis. Table VII-12 presents a summary of the selected performance measures and costs of the best reactive and predictive control strategies. From this table we see that the best performance from the reactive strategies is derived from Alternative III-B (Minimum Storage Constraints) and IV-C (Seasonal and Minimum Storage Constraints). However, Alternative IV-C shows definite cost reductions and, hence, is the preferable reactive strategy.

Strategy V-E, which combines a seasonally constrained reactive strategy with perfect rainfall prediction, demonstrates the potential improvement in performance if rainfall prediction is without error. It shows modest improvements in treatment as demonstrated by the pollutant reduction estimate and a decrease in the average number of times per year the wet weather facilities would have to be placed in operation. These modest improvements occur without any increase in the number or volume of overflows.

Using the present rainfall prediction model (Alternative V-G*), increased the number and volume of overflows. This increase indicates there are errors in the prediction, especially in moderately large storms. In other words, the storms that almost cause overflows under the Basic Operating Strategy (I), are overflowing in the predictive strategies because we could not forecast them accurately.

In summary, it appears that a reactive, seasonally adjusted minimum storage control strategy is the most cost-effective strategy for the NSOC Project as it was evaluated. The latter phrase is emphasized because there have been recent design changes and a change in the overflow objective (from one to four per year) that could have an impact on this conclusion. As the number of allowable overflows increase, the types of storms that must be controlled (or predicted) change. This change may or may not make predictive strategies more attractive. It is important to note that the best control strategy is dependent on both the system's performance objectives and design.

TABLE VII-12

COST-EFFECTIVENESS COMPARISONS OF AUTOMATIC CONTROL STRATEGIES FOR THE NSOC PROJECT

Performance Measure	REACTIVE				PREDICTIVE	
	Strategy I	Strategy II-C	Strategy III-B	Strategy IV-C	Strategy V-E	Strategy[2] V-G*
Overflows						
No/yr.	0.9	1.1	1.1	1.1	1.1	1.4
MG/yr.	9	15	11	13	11	16
Primary Treatment						
Events/yr.	104	97	40	47	39	38
MG/yr.	1,196	1,144	929	954	911	884
Days/yr. [1]	365	212	365	212	212	212
Secondary Treatment						
MG/yr.	7,311	7,357	7,576	7,549	7,593	7,603
Pollutant Reduction (%)	-	0.7	7.4	6.8	7.6	7.9
Cost ($/yr.)	$191,100	<$141,100>	$193,100	<$141,100>	<$124,000>	<$124,000>

[1] Period wet-weather plant must be operational.

[2] Evaluated using 28-year rainfall record; all others used 70-years.

VIII. CONCLUSIONS AND RECOMMENDATIONS

Our conclusions and recommendations relate to the general concepts and methods developed in the course of this study for analyzing, evaluating, and implementing real time automatic control. Although they are not meant to be specific to the San Francisco NSOC Project, the experience gained with this demonstration vehicle provided the basis for the conclusions and recommendations. Consequently, the conclusions and recommendations should not be considered as applicable to all combined sewer systems.

It should also be recognized that the NSOC Project had several unique features that limited the potential for improvements in system performance by automatic control. These limitations include the following: 1) all storage was at the terminous of the project and hence the system as designed would minimize overflows, 2) there was only one pump station to control outflow from the storage facilities, and 3) the overflow objective of one per year was extremely restrictive and required virtually all storms to be completely controlled by storage and treatment.

CONTROL SYSTEM

Conclusions

1. Real time control systems for combined sewer control can be a potentially cost-effective solution to urban stormwater management problems. Automatic control can be more cost-effective than manual control.
2. The objectives and design of a control system are dependent on, and must be responsive to, such factors as constraints imposed by existing and proposed facilities, receiving water characteristics, budget constraints, and applicable laws and standards.
3. Control system studies can develop valuable information relating to the design of the facilities to be controlled, resulting in significant savings in construction and operation and maintenance costs.

Recommendations

1. Control system studies for projects which involve the design of new facilities should be conducted concurrently with the facilities design.
2. The objectives of the control system should be identified as early as possible in the planning process to guide the direction of both the control system and facilities designs.
3. The feasibility and cost-effectiveness of real time automatic control should be demonstrated in the field since practical experience in this area is lacking.

CONTROL STRATEGIES

Conclusions

1. Simple reactive control strategies incorporating a minimum of control hardware and software can result in significant cost-effective improvements in system performance. Such strategies will be particularly effective in the following cases:

    - The seasonal variations in inflows or receiving water assimilative capacity are significant.

    - Separate wet weather treatment facilities are involved that may be shut down for certain periods with resultant cost savings.

    - The physical facilities incorporate or can be modified to incorporate flexibility in the form of storage capacity transfer facilities, and variable treatment rates and levels.

    These and other factors which vary with each application play a major role in determining the best strategies and potential benefits from an RTAC system.

2. Integrated reactive-predictive control strategies appear to hold promise, but their cost-effectiveness is highly sensitive to the following factors:

    - The design of the physical facilities, including size and flexibility as they affect the types of storms providing overflows.

- The objectives of the control system.
- The accuracy of the predictions of rainfall and runoff, which are partly dependent on the local characteristics of rainfall and runoff, as well as the ability to obtain reliable real time data.

Recommendations

1. Reactive control strategies should be more thoroughly investigated to develop optimum reactive strategies.
2. Research in predictive strategies should be focused on developing improved techniques for rainfall prediction.

RAINFALL PREDICTION

Conclusions

1. Rainfall prediction was shown to have modest potential for improving system performance.
2. For the rainfall prediction model developed in this project, errors in the predictions were large enough to prevent the potential performance improvement from being achieved.

Recommendations

1. Research continue in the general area of short-term (six hours and less) rainfall prediction.
2. Techniques other than statistical models be investigated; such techniques might include both radar and satellite photography.

MATHEMATICAL MODELS

Conclusions

1. Models are needed for predictive-reactive or wholly predictive strategies.

2. Models similar to the RUNOFF block of SWMM can be used in real time control systems for predicting runoff from either measured rainfall or predicted rainfall.
3. Neither TREAT nor the TRANSPORT block of SWMM are adequate for real time control.
4. TREAT or similar models can be effectively used to evaluate alternative control strategies.

Recommendations

1. Additional research be oriented to relatively simple transport models that are reliable and that could be used in those cases where predictive strategies prove useful.